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# BUILDING STRUCTURES IN EARTHQUAKE COUNTRIES.

BY

ING. ALFREDO MONTEL.

TRANSLATED FROM THE ITALIAN, WITH ADDITIONS
BY THE AUTHOR.

With 42 Diagrams in the Text and one Plate.

LONDON:
CHARLES GRIFFIN & COMPANY, LIMITED,
EXETER STREET, STRAND,
1912.

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# PREFACE TO ENGLISH EDITION.

I HAVE profited by the translation of my Italian book to extend some parts of it, and especially to give some particulars of the earth- and sea-quake of Calabria and Sicily, 1908, which were not known at the time of the first publication of this treatise.

I am grateful to the scientific and technical world at home and abroad for its favourable reception of my Italian book, and I hope that, with the present edition, published outside of Italy, my modest work will prove in earthquake-shaken countries of some utility to architects, contractors, and others interested in the difficult problem of building in these regions.

A. MONTEL.

ROME, *March* 1912.

# PREFACE.

IN this little work I mainly deal with the problem of building houses which are proof against earthquakes, and in so doing I have made special use of some important works on seismology by Professor F. Omori of the University of Tokyo.

Above all, I have paid attention to the construction of houses in brick and reinforced concrete. The calculations are purposely expressed in such a way as to facilitate their application in analogous cases.

My inducement to write this book has been the recent disastrous earthquake in Calabria and Sicily. In these countries, which so terribly suffered under the misfortune, it is in the highest degree desirable that in the work of rebuilding greater foresight and more scientific methods shall be used than has been the case in the past. And if these few pages should induce others to write more important treatises, from which still better rules for building in seismic countries might be derived, I should consider myself amply rewarded for this modest work of mine.

A. MONTEL.

ROME, *May* 1909.

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# BUILDING STRUCTURES IN EARTHQUAKE COUNTRIES.

## CHAPTER I.

## ON EARTHQUAKES IN GENERAL.

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CERTAIN regions within the terrestrial globe are, for reasons the investigation of which would be beyond the scope of this treatise, subject to convulsions and transformations; and the shocks which result therefrom are transmitted across the earth to distances which are sometimes very far from the point of their origin. These shocks constitute the so-called "earthquakes."

The mode of propagation of the often enormous forces which are in action, and which are called "seismic forces," is very complicated, and the branch of science devoted to them ("seismology") is, as yet, far from having fully explained them.

It has, however, been established—but only by way of gross approximation—that these forces are transmitted across the earth in the same manner as across an elastic body, that is to say, in the form of elastic waves; only, the waves which are here in question are longer than one kilometre, and the earth, or at least its crust, is a by no means homogeneous body in which strata of greater width than one kilometre are rare. The waves must therefore undergo transformations and alterations before they arrive at the surface of the earth. Travelling, as they do, across strata of heterogeneous elasticity and density, they are generally neither purely longitudinal nor purely transversal, but of a complex nature—a mixture of the two. Nor is their velocity always the same, but it varies according to the strata across which they are propagated. Indeed, that velocity is determined by the relation between the modulus of elasticity and the density of the body which the waves traverse—relations which vary from one kind of rock to the other.

From experiments made, it seems to result that the modulus of elasticity generally grows more rapidly than the density whenever the density of the rocks increases. Thus it happens that the elastic waves are propagated with greater velocity in the deep levels of the earth's crust than at its surface, where the less dense rocks occur.

The study of the velocity of propagation of these waves also enables us to advance hypotheses regarding the inner constitution of the globe. Thus, Omori has found that the velocity of the first waves indicative of an earthquake—those which travel in the deeper regions of the earth—is about 13 km. per second, which would denote a mean of propagation of the density $\rho = 3{\cdot}5$, and of a modulus of elasticity $E = 6{\cdot}10 \times 10^{12}$ C.G.S.

On the other hand, the waves which travel along the surface of the soil have a mean velocity of 3·3 km. They constitute the more important part of the earthquake—that part which produces the greatest effect, convulsions of the soil, the destruction of buildings, etc.[1]

According to Omori, three periods can generally be distinguished in an earthquake:

Initial tremors.
Principal period.
Final period.

The first period is represented by the waves which travel at great depth; of these the most rapid ones arrive first, and the others gradually after.

The principal period is represented by the surface waves, and to it are due the great effects of the earthquake.

The final period is formed by the expiring forces of the first cause of the earthquake (which we do not investigate here), and by the *vis inertiæ* as well as by the momentum of the masses in the movement, which require some time before they return to the state of repose.

From the data collected by Omori[2] with reference to nineteen destructive, semi-destructive, or strong earthquakes which have occurred in Japan, it results that the duration of the most violent part of the principal period in destructive shocks is generally from four to ten seconds. If, however, the earthquake is very great, *i.e.* extended and violent, the duration may be prolonged up to thirty seconds. These data may be useful in studying the destructive effects of an earthquake upon buildings.

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[1] *Cf.* H. Nagahoka, *Publications of the Earthquake Investigation Committee in Foreign Languages*, No. 4, Tokyo, 1900, p. 47; F. Omori, *On Seismic Instruments*, p. 248 (from the Transactions of the first International Seismologic Conference, Strasburg, 1901).

[2] *Bulletin of the Imperial Earthquake Investigation Committee*, vol. ii. No. 2.

The earthquake must necessarily have also a direction. In the region of the earthquake this direction is generally uncertain; it varies from place to place (owing probably to the heterogeneous structure of the earth's crust), and also from one moment to the other. There is, however, generally a *main direction* of the movement, which is that of the greatest horizontal movement.

It may be pointed out here that there are no rotary or vertical earthquakes. Sometimes one meets objects like columns, obelisks, etc., which under the action of an earthquake have undergone a rotatory movement, but that does not by any means prove that the force which produced it has been a rotatory one. Thus, if an object on the point of sliding is impeded by an obstacle located excentrically in relation to it (for instance, if a portion of the superficies of a base situated near the periphery could not slide owing to attrition), the object can evidently be subjected to a rotatory movement even if the force which moves it has a constant horizontal direction.

For the purposes of the present treatise we shall investigate only the principal period of the earthquake and the surface waves.

It is yet a controversial point whether these waves denote also a vertical movement or only a horizontal one. Omori denies the vertical motion,[1] his reason being that the tests made by him for this purpose have not proved its existence. On the other hand, the majority of seismologists, and also the physicists Kelvin and Rayleigh, assert the necessity of the vertical component. It may therefore be assumed that this component exists, but that, compared with the horizontal one, it is very small and difficult to observe.

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[1] De Montessus de Ballore, *La Science Seismologique*, Paris, 1907, p. 367.

This point is, however, of no great practical importance. Even though the surface-waves be devoid of a vertical component, it remains a fact, corroborated by Omori himself, that a vertical component is likewise met in earthquakes, which would mean that this vertical movement is produced by some other cause. The vertical component is, however, generally small when compared with the horizontal one, and, moreover, as Omori points out, it could not produce any important effects upon buildings even if it were great. We shall therefore not insist upon this question, and in our treatise we shall without exception assume that the action of an earthquake in a point $P$ of the earth's surface is a movement of going and coming in a horizontal direction.

Let us consider what will be the action of the point $P$ during the oscillation of the soil.

It starts from the position of repose $O$ (fig. 1), where its acceleration is zero, and travels to $Q'$, where the acceleration reaches its maximum value $A$. From there it returns and repasses through $O$, where the acceleration is again zero while the velocity of the movement is at its highest. Then it will go to $Q$, where the acceleration will again have a new maximum value $A$ and the velocity will be zero, and so on. $OQ = OQ'$ represents by way of diagram the amplitude $a$ of the oscillation (*i.e.* one-half of the extension of the movement of the soil); if, then, $T$ is assumed to be the total duration of the movement, we get the formula $A = \frac{4\pi^2 a}{T^2}$.

FIG. 1.

Earthquakes may be classified either according to the maximum acceleration, or according to the extension of the movement, or according to its duration. It is not stated, however, that the most widely extended earthquakes always produce the greatest effects, nor that those of the shortest duration do that; on the contrary, the latter are generally also the weakest. What really constitutes the importance of an earthquake is its maximum acceleration, and it is on the basis of that criterion that Omori proposes his scale of destructive earthquakes. This scale we give hereafter *in extenso*; it indicates the relation between the maximum acceleration of the earthquake and the effects produced by the latter. It will be remembered that he is dealing with the horizontal movement, which is generally admissible.

Professor Omori observes later on (*Bulletin, etc.*, vol. iv. No. 1) that even though the movement had a very strong vertical component, *i.e.* that it strongly emerged above the horizon, the value of its maximum acceleration would not differ in a practically perceptible manner from the maximum horizontal acceleration capable of producing effects of convulsions equal to those due to the co-existence of a vertical and a horizontal component of the acceleration. The values of the maximum acceleration (understood to be horizontal) which have been chosen by Omori for his earthquake scale were derived from the examination of bodies overturned by the earthquake of Mino Owary on the 28th October 1891. If we call that maximum horizontal acceleration $a$, it was found for Nagoya and Gifu that $a$ was $= 2600$ mm. per sec.$^2$ and $= 3000$ mm. per sec.$^2$ respectively. Judging, however, by the seismographic data ascertained at the meteorological observatory of Nagoya, the amplitude of the vertical component $a''$ was apparently about one-third of the horizontal $\alpha'$, which means that the direction of the movement emerged by an angle of 18° 26′ over the horizon. If we call the maximum acceleration of this movement $\alpha_0$, it results from Omori's calculation that for the two above-named localities $\alpha_0$ was equal to 2518 mm./sec.$^2$ and 2869 mm./sec.$^2$ respectively. The difference $\alpha - \alpha_0$ is, therefore, 82 and 131 mm./sec.$^2$ respectively. Moreover, if we assume that the angle of emersion of the movement above the horizon had been 45°, there would have been for Nagoya and Gifu $\alpha_0 = 2906$ mm./sec.$^2$ and 3208 mm./sec.$^2$ respectively, which would have resulted in the difference $\alpha_0 - \alpha$ being 306 and 208 mm./sec.$^2$ respectively, or $\frac{1}{8}$ and $\frac{1}{12}$ of $\alpha$. It may therefore be safely assumed that practically the values of $\alpha$ given by Omori in his scale do not differ from those of $\alpha_0$ even in cases where the emersion angle is considerable.

Omori's scale has been compiled with special regard to Japan.

## OMORI SEISMIC SCALE.

I. Maximum Acceleration = 300 mm. per sec. per sec.—The shock is rather strong, so much so that it generally induces people to escape from their houses into the open. The walls of badly constructed brick houses crack slightly and some parquet falls down; ordinary wooden houses are shaken in such a degree that they loudly creak; furniture is overturned; trees are visibly shaken; the water in ponds and pools gets turbid, owing to the disturbance of the mud; pendulum clocks stop; some very badly built factory chimneys are damaged.

II. Maximum Acceleration = 900 mm. per sec. per sec.—The walls in the wooden houses of Japan crack; old wooden houses get slightly out of plumb; the Japanese tombstones and the badly constructed stone lanterns are overturned; in a few cases the flow of the thermal and mineral springs is changed; ordinary factory chimneys are not damaged.

III. Maximum Acceleration = 1200 mm. per sec. per sec.—About one-fourth of the factory chimneys are damaged; badly constructed brick houses are partially or totally destroyed; some old wooden houses are destroyed; wooden bridges are slightly damaged; some tombstones and stone lanterns are overturned; Japanese sliding doors (covered with paper) are broken; the tiles of wooden houses are displaced; some fragments of rocks are detached from the sides of the mountains.

IV. Maximum Acceleration = 2000 mm. per sec. per sec.—All factory chimneys are ruined; the majority of the ordinary brick houses are partially or totally destroyed; some wooden houses are totally destroyed; the wooden sliding doors are mostly thrust out of their channels; crevices from 2 to 3 inches (5 to 7½ cm.), wide appear in low and soft grounds; here and there the embankments are slightly damaged; wooden bridges are partially destroyed; ordinarily constructed stone lanterns are overturned.

V. Maximum Acceleration = 2500 mm. per sec. per sec.—All ordinary brick houses are very seriously damaged; about 3 per cent. of the wooden houses are totally destroyed; some Buddhist temples are ruined; the embankments are badly damaged; the railways are slightly contorted; ordinary tombstones are overturned; brick walls are damaged; here and there, large fissures from 1 to 2 feet (30 to 60 cm.) wide appear along the banks of the watercourses. The water of rivers and ditches is thrown on the banks; the contents of the wells are disturbed; landslides occur.

VI. Maximum Acceleration = 4000 mm. per sec. per sec.—The greater part of the Buddhist temples are ruined; from 50 to 80 per cent. of the wooden houses are totally destroyed; the embankments are almost destroyed; the roads through paddy fields are ruined and interrupted by fissures in such a degree that traffic by animals or vehicles is impeded; the railways are very much contorted; great iron bridges are destroyed; wooden bridges are partially or totally damaged; tombstones of solid construction are overturned; fissures some feet wide appear in the soil, and are sometimes accompanied by jets of water and sand; iron or terra-cotta tanks embedded in the ground are mostly destroyed; all low-lying grounds are completely convulsed horizontally as well as vertically in such a degree that sometimes the trees and all the vegetation on them die off; numerous landslides take place.

VII. Maximum Acceleration = much more than 4000 mm. per sec. per sec.—All buildings are completely destroyed except a few wooden constructions; some doors or wooden houses are thrown over distances from 1 to 3 feet; enormous landslides with faults and shears of the ground occur.

We also reproduce the Mercalli scale, which was adopted in 1900 by the Geodynamic Department of Italy.

## MERCALLI SEISMIC SCALE.

I. Instrumental Shocks, *i.e.* shocks merely recorded by the seismic instruments.

II. Very Slight Shock, observed only by persons in a state of perfect quiet, especially on the upper floors of houses, or by very sensitive and nervous people.

III. Slight Shocks, observed by several, but only comparatively few persons among the inhabitants of a given region; people say, "It was scarcely felt," without any apprehension, and generally without having noticed that it was an earthquake, until after other people mentioned that they had also observed it.

IV. Perceptible or Moderate, noticed not generally, but by many persons in the interior of houses, though on the ground floors by a few only—without alarm, though fittings and glassware tremble, the ceilings creak, and suspended objects oscillate slightly.

V. Strong, observed generally in the houses, but in the streets by a few people only; sleeping persons are awakened, some with a feeling of alarm; there is banging of doors, sounds of bells, rather strong oscillations of suspended objects, stopping of clocks and watches.

VI. Very Strong, observed by everybody in the houses, and by many with fright; flight to the open ground; fall of objects in the houses; collapse of chimneys, with some slight cracks in less solid houses.

VII. Most Strong, observed with general terror and flight from the houses; perceptible also in the streets; sounds of church bells; falls of chimneys and tiles; numerous cracks in the buildings, but generally only of a slight nature.

VIII. Ruinous, observed with great terror; practical collapse of some houses, with general and considerable damage to others; without human victims, or only with some isolated personal misadventures.

IX. Disastrous, with total or almost total destruction of some houses, and great damage to many others, so much so that they are rendered uninhabitable; human victims not very numerous, but spread over various points of the inhabited places.

X. MOST DISASTROUS, with destruction of many buildings and many human victims; crevices in the soil; landslides in the hills, etc.

In judging the intensity of the shocks, account must be taken rather of the aggregate total of the damages and the ruin caused by them, than of some isolated facts which may often be due rather to the particular condition of some building than to the intensity of the shock; and, in particular, it must be considered whether at the moment of the earthquake the inhabitants were in the houses or in the streets, or assembled in churches or theatres.

We append a comparative table of the Omori and the Mercalli scales. In compiling it we have taken account of the relation between the Omori scale and the Rossi Forel scale,[1] and between the Rossi Forel scale and the Mercalli scale.[2]

| Omori scale. | | Mercalli scale. |
|---|---|---|
| I. | 300 mm. sec.$^2$ | VI. |
| II. | 900 ,, ,, | . . |
| III. | 1200 ,, ,, | VII. |
| IV. | 2000 ,, ,, | VIII. |
| V. | 2500 ,, ,, | IX. |
| VI. | 4000 ,, ,, | X. |
| VII. | >4000 ,, ,, | |

In order to give a clear idea of what an earthquake is, we reproduce the part which possesses the greatest interest for us of the description of the Tokyo earthquake on 20th June 1894 :—

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[1] F. Omori, *Publications, etc.*, No. 4, p. 140.

[2] De Montessus, *loc. cit.*, p. 51.

[3] T. Sekiya and F. Omori, *Publications, etc.*, No. 4, p. 35.

"That earthquake made itself felt over a zone of 24,000 square miles, and 26 persons were killed in it, while 171 were seriously injured. In Tokyo there were no houses totally destroyed, but in the low-lying quarters of the town several brick buildings were seriously damaged, and tombstones as well as stone lanterns were overturned, small cracks appeared in the ground, and some jets of water rose from the earth.

"*Horizontal Movement.*—The earthquake began, as usual, with tremors which lasted 10 seconds. The movement had reached an amplitude of about a millimetre when the instruments commenced to indicate it. We shall take that instant as the beginning of the earthquake. The movement, already strong in the 1st and 2nd seconds, became, of a sudden, violent, and the soil was moved 37 mm. during the interval between the 3rd and 4th seconds. This was followed by a movement of 73 mm. in the opposite direction, which was the greatest horizontal one during the earthquake, and was again followed by a motion of 42 mm. During about a minute after these three strongest shocks the movement grew very much weaker, although it was still large in extent. A few great oscillations occurred between the 40th and the 53rd seconds, and again between the 70th and 78th, but the intensity of the shocks was no longer so strong as at first. The relative smallness of the damage caused by this earthquake notwithstanding such great horizontal motions is, no doubt, due to the small number of violent oscillations.

"*Period of the Horizontal Movement.*—The maximum horizontal movement, mentioned above, occurred in 0·9 second, so that the complete period of the oscillation would be 1·8 seconds.

"*Direction of the Movement.*—The direction of the movement changed, as usual, during the earthquake, but the greatest horizontal shock was directed towards S. 70° W., and the principal movements before and after had also practically the same, or else the opposite, direction. We have examined the direction in which, in different parts of Tokyo, 245 stone lanterns were overturned, and found the mean direction to have been S. 71° W. Thus it is seen that the direction of the overthrow was identical with that of the greatest horizontal movement.

"*Vertical Motion.*—The greatest vertical movement was of 10 mm., and occurred during the 3rd second, almost simultaneously with the first strong horizontal motion. There were some more or less important vertical movements during the next 30 seconds.

"*Maximum Acceleration.*—The maximum acceleration of the movement of a particle of the soil, calculated according to the value of the maximum horizontal movement and its duration, is 444 mm. per sec. per sec. That was the maximum acceleration in the high quarters of Tokyo, where the soil is compact clay; in the low-lying quarters, where the ground is soft and marshy, it almost reached the value of 1000 mm. per sec. per sec. Now, the maximum acceleration is the measure of destructive power of an earthquake, and from that it must be deduced that whenever it reaches the above values the chimneys will be greatly damaged and the buildings shaken, as they were on this occasion. As to the great earthquake of Mino Owary in 1891, it has been calculated by the examination of numerous overturned and fractured bodies that the maximum acceleration of the earthquake in the megaseismic region has been from 3000 to nearly 10,000 mm. per sec. per sec.

"*Duration of the Earthquake.* — The shocks lasted about 4 minutes and 30 seconds."

We shall now refer to the disastrous earthquake of Calabria and Sicily on the 28th December 1908, during which, however, in face of the enormity of the disaster, which destroyed every means of observation, no scientific investigations could be made. The movement was perceptible within a radius of 200 km., but destruction of, or grave damage to, buildings only occurred within a much smaller zone of about 30 km. in length and 20 km. in width, including Messina, the Straits, Reggio Calabria.

According to the evidence which has been collected,[1] it seems that at Messina the duration of the earthquake was from 25 to 27 seconds, and that it had various phases. The first, eminently like an upward shock and, lasting from 2 to 3 seconds, does not seem to have been very strong or to have caused direct damage. This phase was apparently followed by an undulatory movement, in the normal sense, at the Straits of from 7 to 8 seconds, which caused the fall of pieces of mortar, of bricks, and of some less solid and projecting parts of buildings. After a very short interval of about 1 second most violent undulations, the direction of which was perpendicular to the first ones, commenced, and lasted 15 seconds. Within that phase the destruction of the city seems to have occurred.

At Reggio Calabria it appears that the earthquake lasted about 35 seconds, and that the movement was at first like an upward shock or upheaval, and then assumed an undulatory character, during which phase it caused the destruction of the city. According to other witnesses the movement was at first undulatory and then stronger, rotatory. It ought to be mentioned that in the most convulsed zone of the earthquake the survivors state that they have felt a gyratory movement which produced nausea and giddiness, and that innumerable objects were subjected to a gyratory movement. But these phenomena do not, as we have already explained, imply the existence of a rotatory earthquake.

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[1] M. Baratta, *La Catastrofe sismica Calabro-Messinese*, p. 268.

According to Professor Mercalli, about 98 per cent. of the houses in Messina and Reggio collapsed totally or partially.[1] The streets of the two cities, generally narrow, were covered with the fragments of ruined houses, many of which had been 15 and 20 metres high. Thus many people who had not perished in the houses lost their lives in the streets while trying to save themselves. According to the most credible reports, about 55,000 out of a total number of 90,000 inhabitants lost their lives in the city of Messina, and in the city of Reggio about 9000 or 10,000 out of a total number of 27,000.

According to the estimate of Omori,[2] the maximum acceleration of the Messina earthquake was about 2000 mm./sec.$^2$ At Nagoya, during the earthquake of 1891, it had been 2600 mm./sec.$^2$, but out of a total population of 165,000 only 190 perished. This shows that the death of so many people in Messina, Reggio, etc., was primarily due, from the seismologic point of view, to the completely unsuitable construction of the buildings in these cities.

We shall conclude this chapter with a few words on sea-quakes.

The sea-quake, or tremors of the sea, is nothing but the effect of convulsions and shocks of the bottom and the shores of the sea upon the great mass of the water. This phenomenon has, of course, been less closely studied than the earthquakes proper, because the sea is much less densely populated than *terra firma*. It has, however, been ascertained that a sea-quake does not generally affect ships on the high seas[1] in any degree, because the mass of the water forms a very strong cushion which softens the shocks and the effect of the convulsions of the solid crust of the earth. On the other hand, a sea-quake may be very strong in the neighbourhood of the coast.

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[1] *Report of the Parliamentary Sub-Committee on the Condition of the Country People in the Southern Provinces and in Sicily*, vol. v. section iii.

[2] *Bulletin, etc.*, vol. iii. No. 2.

Often an earthquake and a sea-quake will simultaneously trave along a sea-shore, and the damage caused by the enormous waves of the latter sometimes exceeds by very far that produced by the former. The height of the waves during a sea-quake may attain some dozen of metres; their velocity is greater than that of the waves during the most violent storms, and they occur again and again at variable intervals up to periods of some hours.

The earthquake in Calabria and Sicily of the 28th Decembe 1908 was accompanied by a strong sea-quake.[2] The latte originated in the Straits of Messina and made itself felt violently on the shore at different points of the coast during from 4 to 10 minutes after the earthquake. By the almost unanimous consensu of expert opinion, it was preceded by the retreat of the sea-water. There were at least three heavy swells like tidal waves.

The destructive effects manifested themselves along the Calabrian coast over about 40 km., and along the Sicilian coast over about 100 km. At one of the points where the damage was more considerable, between Pellaro and Lazzaro, the force of the water broke down the iron girders of a railway bridge of 42 metres, and removed the sandy beach over a maximum width of about 100 metres. This latter phenomenon, which was observed in various places on both shores of the Straits, seems, however, to have been combined with a sagging or subsidence of the soil. In some places the force of the waves of the sea-quake was so strong that it broke down houses, and numerous people lost their lives owing to it.

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[1] De Montessus, *loc. cit.*, p. 200.

[2] Omori, *Bulletin, etc.*, vol. iii. No. 2; Baratta, *La Catastrofe sismica Calabro-Messinese*, p. 363.

According to G. Platania and Omori, the height of the waves exceeded on some points of the coast 10 metres, and in many others 7 metres. Omori states as his opinion, that that sea-quake was to a certain extent caused by the transmission of the seismic energy from the solid crust of the earth to the water of the Straits, but especially by the subsidence of the bottom of the sea in that region.

# CHAPTER II.

## SEISMIC ACTION AND NATURE OF THE SOIL.

Compact Soil—Loose Soil—Marginal Vibrations—Declivities—Junction Lines be Different Soils—Gravity Waves—Their Length, Duration, Velocity—Formation—Seismic Action in Soft Ground—Seismic Action in Deep Wells—Selection of Ground on which to Build.

CLOSE observation of earthquakes has proved that during the seismic action the soil, even within a short space, may be under the action of forces having different intensity, direction, and phases from one point of the soil to the other. If, therefore, its cohesive capacity is not sufficiently great, the soil is liable to the formation of crevices and to disintegration.

That does not generally occur when compact rocks are in question, but it may very easily take place in the friable and not very homogeneous embankments, and the crevices generally appear along the lines of least cohesion, *i.e.* of least resistance of the mass.

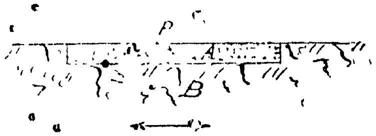

FIG. 2.

In fig. 2, *B* represents an elastic and compact mass of ground, and *A* a layer of friable filling. If an oscillating movement approaches in the direction indicated by the arrow, the particles composing the layer *A* are put into motion, and according to the law of *vis inertiæ* they will have a tendency to change their position, while the layer *A* will tend to extend itself horizontally, and therefore to disintegrate.

An obstacle to the formation of crevices are the solid ramparts of the mass *B* by which the layer *A* is enclosed. In case one or more of these ramparts should give way, so that the support from that side failed, the layer *A* would more easily disintegrate, and there would be a stronger tendency to the formation of fissures. The same would be the case if the mass of the layer *A* were weakened by rivers, canals, or excavations which traverse it. It is presumable that, parallel to these lines of weakness, fissures would arise owing to the subsidence of the friable mass, which during the shocks would find no support on the part of the line of weakness. And that has been corroborated by experience. Thus, Sekiya[1] found on the highest summit of a steep declivity (ground clayey), at a height of 38 feet from its base, the amplitude of the oscillation had been doubled owing to the absence of support for the soil from the side of the declivity.

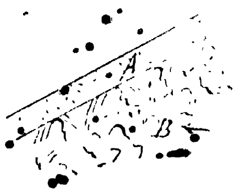

Fig. 3.

Fig. 3 represents a declivity with agricultural filled-up and embanked soil on a mass of compact rock. It is obvious that, in case of telluric motions in the direction indicated by the arrow, the layer *A* would not only tend to disintegrate, but also to be detached from *B*. In case its adherence to *B* should fail, and if the inclination is strong enough to over-come the coefficient of friction between $A$ and $B$, there will be a slide of $A$ towards the bottom due to gravity. Other conditions being equal, this danger will, of course, increase in the same ratio as the declivity of $B$.

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[1] F. Omori, *Note on Applied Seismology*, Part I. p. 345 (from the Transactions of the first International Seismological Conference, Strasburg, 1901).

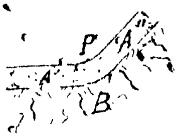

FIG. 4.

Fig. 4 represents a piece of embanked land at the foot of a hill. The point $P$ where the sloping part of the ground meets the level part is evidently much exposed to the effect of seismic action. In view of the difference in height between the portions $A'$ and $A''$ of the embanked land—in view also of the contour form of $B$—it may be expected that $A'$ and $A''$ will vibrate differently from one another. Moreover, $A''$ is liable to slide along the decline. All this renders the position of $P$ very precarious.

Generally speaking, it may be saidthat a given piece of ground will feel the effect of an earthquake very strongly when it is on the point of junction between grounds which behave differently under a shock. Whenever different kinds of soil are in question this is true also, even when everything else is disposed as shown by fig. 2, which, regarding uniformity of width and everything else, obviously represents the most favourable condition for filled-up and embanked land.

Let us consider further and in detail the case represented by this figure. Suppose the mass $A$ to be extremely soft and plastic—nay, let us put an extreme case, and say it is liquid. Shocks which alternately agitate in a horizontal direction the basin containing this liquid will have the effect of producing waves on the surface of the liquid mass, the wave-producing process being very much more difficult as the liquid is denser and "pastier." Well, experi ence has established the fact that, during an earthquake in very soft, diffused, sandy or marshy ground, exactly similar waves can be observed, whose crests may be raised to considerable heights above the level of the soil. They[1] sometimes have, according to Oldham, a mean length of 30 feet and a height of 1 foot. Sometimes when, after the shock was over, the earthy mass was not sufficiently plastic to return to its primitive position, the waves remained impressed upon the soil. Assuming that $T$ means the period, $V$ the velocity, and $l$ the length of such waves, there are, according to Omori,[2] good reasons for asserting that $T$ may be one or a few more seconds, and $l$ some 10 metres or more. Assuming $l = 20$ m., and $T = 5$ seconds, we get $V = \frac{l}{T} = 4$ metres per sec.

But we can easily form a fairly clear idea of these waves, if we imagine a basin full of water and shaken. We will then understand that the crests of these waves must be near to one another (and that means short waves), and on the surface, which means that in very little depth there is no longer any trace of them; it will also be understood that in the case of plastic matter of considerably greater density than water the waves will be slow. Numerous data[3] confirm this theory.

The production of such waves will only be possible in ground whose form can be altered without great force. In all other kinds of ground, it is obvious from what we have said above—and as has been pointed out by Dutton—that the formation of such waves is hindered and impeded by the fact that, in order to mould the mass in such a manner, enormous forces and very great intermolecular activity would be required. In such cases the energy communicated to the soil by the seismic shocks will find easier ways to spend its strength (further propagation, production of landslides, etc.).

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[1] De Montessus, *loc. cit.*, p. 441.

[2] F. Omori, *On Seismic Instruments*, p. 251 (from the Transactions, etc.).

[3] De Montessus, *loc. cit.*, p. 437.

It is also easy to imagine the disastrous effects which, in view of their great displacement, these waves—called waves of gravity—may have upon buildings: a reason why building on soft ground ought to be absolutely avoided.

As we have already seen in the description of the Tokyo earthquake on 20th June 1894, the maximum horizontal acceleration is, other conditions being equal, greater on soft than on hard ground. This is explained by the reciprocal molecular displacements to which soft, pasty ground is subjected by the *vis inertiæ* under the impulse of the surface waves constituting the principal period of an earthquake.

The fact that the waves of gravity are exclusively surface waves, and accompanied—they are even caused—by horizontal movements of the soft ground, makes us believe that at a certain depth, where the waves of gravity have ceased to exist, the horizontal movements of the ground, due to its plasticity, must be smaller than at the surface. Extending our argumentation a little further, we would come to the conclusion that also in non-plastic ground the reciprocal displacements of the particles of the ground, and therefore the oscillations, are smaller at a certain depth than on the surface of the soil.

That is what really occurs, and from the observations of Japanese seismologists it results that at the bottom of wells some

7 or 8 metres deep the motion of the rapid waves recorded by the seismic instruments was much weaker than at the surface. This proves, according to Omori, that the effects of destructive earthquakes are slighter in deep wells than at the surface.

We have, hitherto, touched only the most important of the principal effects of earthquakes on various kinds of soil. Many others might be mentioned, such as swellings of the ground, vast upheavals, deep precipices, which affect not merely the surface strata; and in addition to these, the formation of small craters, jets of water and sand, and so on. All these phenomena can be explained when the constitution of the earth's crust and the various elements of which it is composed are duly considered; but this does not come within the scope of our treatise.

A most important point, intimately connected with the locality as well as the nature of the soil, and which ought to be mentioned here, is, as seismic history teaches, that an earthquake produces in the same districts always analogous effects. That suggests that the causes, too, must be analogous, and that each of the different seismic regions is bound to vibrate always in the same manner and in the same direction. Just as there are regions which are subject to earthquakes, and others which are immune from them, so each of the former is subjected in a given manner and not in any other.

The short considerations which we have hitherto devoted to the effects of earthquakes with reference to the various kinds of soil furnish us with criteria which must guide us in the selection of a place for building in seismic regions—criteria regarding which, let us say at once, there is complete agreement among all authorities.

In the first place, the seismic history, if there be one, of the district ought to be studied.[1] Note should also be made of the places in the locality where the buildings have offered the greatest resistance, and how they bore themselves during the shocks, how they were located, how they were constructed, their heights, the directions of the walls, etc. Experience teaches that solid and compact soil ought to be selected; in the plains as well as in hilly country, buildings are safest on such ground.

Avoid heterogeneous and friable soil, especially if it is of little thickness and on a steep incline.

Avoid the vicinity of rivers, canals, and interruptions due to geological causes, such as crevices, etc.

Avoid also bases of declivities and, in general, changes in the inclination of the soil.

Avoid soft, pasty, marshy ground, also filled-up and embanked land.

Avoid also the immediate neighbourhood of the sea or of lakes, on account of the agitation of the water by the earthquake.

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[1] For the seismic history of Italy, see M. Baratta, *I terremoti d' Italia*, Turin, 1901.

# CHAPTER III.

# ON SEISMIC BUILDINGS IN GENERAL.

Resistance to the Seismic Force—Ordinary Houses—Materials proposed: Timber, Iron, Masonry, Reinforced Concrete—Advantages and Disadvantages—Houses with Free Walls—Monolithic Houses.

FROM what we have said before, it results that in very solid and compact ground, the soil, under the influence of seismic forces, practically oscillates in a horizontal direction, and that in other kinds of ground there may exist also an important vertical component of the movement.

In aseismic regions (*i.e.* not liable to earthquakes) it is generally sufficient if a building is so constructed that it will withstand the action of gravity, though in certain localities account must also be taken of the force of the wind; but that is generally quite a secondary consideration which merely concerns the roofing.

In countries, however, which are liable to earthquakes, account must also be taken, even in the most favourable circumstances (*i.e.* when the building is constructed on hard and solid ground), of a great horizontally oscillating force. It is therefore only natural that ordinary houses which were not constructed with a view to such a contingency do not always hold out. We take, however, the opportunity of pointing out here that if, in countries subjected to earthquakes, building operations had always been carried out in *a perfect manner, or even only in accordance with the ordinary* rules, there would have been no such very great disasters as the history of earthquakes has had to register, because a good house well constructed of good materials and with perfect workmanship can always offer considerable resistance to the disintegrating action of seismic movements. That has been proved by long experience.

The materials most often proposed for building in seismic regions, or at least as principal elements of construction, are timber and iron.

Timber is very suitable; it is light, elastic, a bad heat-conductor, and its use does not involve any technical difficulties. From these points of view timber is even the ideal material. It has, however, its disadvantages with reference to its preservation, to cleanliness, and, by far the most serious of all, the danger of fire. It is notorious that very often after earthquakes conflagrations break out, and consequently that defect of timber as a "seismic building material" is doubly grave. But at any rate timber can always render excellent service, especially when light constructions are required to be rapidly erected. It is only in cases of that kind that its employment seems really advisable.

Iron is also suitable, but less so than might appear at first sight. It does not offer good resistance to conflagrations, because under the action of fire it softens and gives way; moreover, it is easily corroded. These drawbacks are so much the more serious as iron constructions are very expensive.

As a type of ordinary construction, it would in our opinion be above all advisable to choose masonry in bricks or reinforced concrete, which latter material begins to be widely adopted also in this kind of work. The use of masonry has taken root in Italy to such an extent, and it offers so many advantages, that it is doubtless advisable to attempt to adapt it to the special requirements of the case instead of declaring it unsuitable, or even, as some experts would have it, banishing it entirely from seismic buildings. Certainly masonry—composed, as it is, of rather heavy materials, not very elastic, fragile, and, owing to its being bound together by mortar whose tensile strength is generally uncertain, easily disintegrated—is not the ideal material for this kind of work. Nevertheless it does not seem impossible to obtain, by strict observance of certain standard rules and by employing only the very best material and labour, houses in masonry which will resist earthquakes.

As regards reinforced concrete, the advantages it offers in ordinary constructions are now generally acknowledged, and in the case before us these advantages, viz. perfect security against fire, as well as great strength, lightness, and rapidity of construction, are of even greater importance. That it is perfectly proof against fire has been proved by tests made in several places.[1] The official report of the tests carried out at Gand says:—"The buildings tested were exposed to a temperature of from 700 to 1200 degrees Celsius. It was found that only the parquet had been slightly damaged. The resistance and elasticity of the floors had remained unaltered. As a heat conductor it has been proved to be very slow, for, while one side of a wall 12 cm. wide was in contact with fire, it was possible to hold one's hand against the other side.

Its rigidity is proved, *inter alia*, by the following tests made in Paris by engineers of the Paris-Orleans railway. A weight of 100 kgs. falling from a height of 4 metres produced upon a floor of reinforced concrete vibrations of 1·2 mm. maximum amplitude, which were extinguished in $\frac{5}{7}$ of a second. Upon a floor of iron beams with brick vaults, of about the same width, a weight of 50 kgs. falling from 6 metres produced vibrations of 7·8 mm., which were extinguished in 2 seconds. The weight of the reinforced concrete floor was 300 kgs. per sq. m., and that of the iron floor 480 kgs. This shows how much more rigid reinforced concrete is than iron.

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[1] C. Guidi, *Le Costruzioni in Beton Armato*, Turin, 1907, p. 107.

The great resistance which reinforced concrete offers to static and dynamic forces entails as a natural consequence the lightness of such constructions.

It is not necessary to allude to the other advantages of reinforced concrete, such as freedom from maintenance expenses, best utilisation of the iron, and the latter's perfect protection against rust, since these advantages refer in an equal degree to ordinary buildings of reinforced concrete, and are well known.

As regards the defects of reinforced concrete, they usually arise from the non-observance of the proper rules of construction. To these defects there is, in our case, added another and more serious one, viz. the difficulty of finding workmen competent to do this kind of work, in places which are often, from the economic and industrial standpoint, very little advanced.

When in planning a house its resistance to earthquakes has to be kept in view, two methods may be followed: either the different members of the house may be calculated so that each one resists on its own account; or the house may be planned so that it resists as a homogeneous entity, as a monolith. The second method would seem to be the more reasonable one. In that case it is, however, indispensable that the various parts of the house should be so well bound together as to offer a reasonable certainty that under the influence of seismic forces the whole house will have a common vibration period, and not that each one of the parts which compose it will vibrate on its own account.

We shall see later on when it will be advisable to select one type, and when the other.

In the following chapters we shall first of all describe a Japanese model of light construction in timber, and, passing from this to real houses, study either of the two types mentioned above; for brevity's sake, houses of the first type will be termed "free-wall houses," and those of the second, "monolithic houses." These studies will be applied to brick masonry and to reinforced concrete.

## CHAPTER IV.

# ABSTRACT OF JAPANESE RULES FOR THE CONSTRUCTION OF EARTHQUAKE-PROOF WOODEN HOUSES.[1]

Foundation—Framework—Foundation Sills—Foot-braces—Pillars—Junctions—Roof Roof-frame—Joints—Junction of two Buildings—Materials—Conclusion.

### FOUNDATIONS.

THE material for the foundations may be one of the following three: concrete, broken stone, stone blocks. The concrete may be made of cement or of lime, or of a mixture of the two. Of whichever kind it may be, it is always preferable to broken stone or stone blocks. Concrete made of cement is the best material for foundations.

If the ground is too soft or humid, the first work in making the foundation is to drive a frame of piles into the ground in order to obtain a solid support. Less may be done if the ground is dry. For the foundation to be placed upon the groundwork large flat stones should be selected, and they should protrude above the ground by one-half of their thickness. These stones form the base of the building.

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[1] *Publications of the Earthquake Investigation Committee*, No. 4, p. 1.

## CONSTRUCTION OF THE FRAMEWORK.

In constructing an earthquake-proof house it is deemed advantageous either to erect the pillars upon horizontal beams, called “foundation sills” (fig. 5); or to unite them by means of a beam laid down laterally against the foot of each one of them, and called “foot-brace” (fig. 6); or to use two pieces of timber forming a double pillar, between which the foot-brace runs (fig. 7); or to lay down two foot-braces on two opposite sides of the pillars (fig. 8). If, in the case shown by fig. 5, greater stability is required, a solid junction of the pillar with the foundation-sill may be effected by means of iron clamps or straps. In adopting the method shown by fig. 6, foot-bracing must be used whose thickness is greater than one-third of the thickness of the pillars. A disposition in accordance with fig. 7 requires an extra-strong pillar for each corner of the building, notwithstanding that the pillar may be weakened by the foot-brace. Another method consists in using a quadruple pilaster, as shown in figs. 9 and 10. In that case pieces

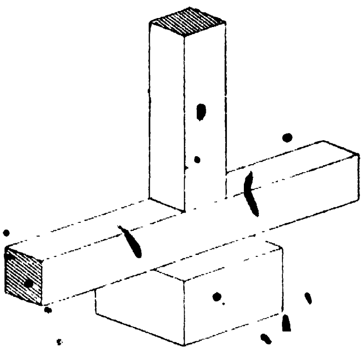

FIG. 5.

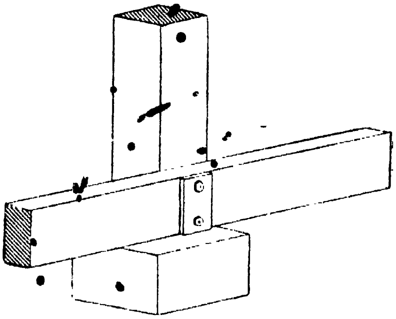

FIG. 6.

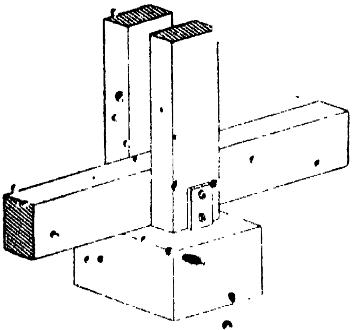

FIG. 7

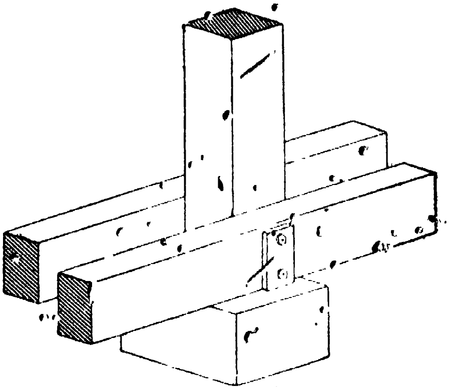

FIG. 8.

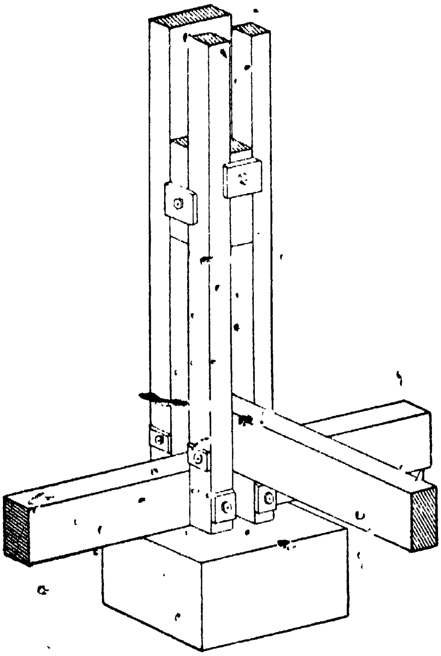

FIG. 9.

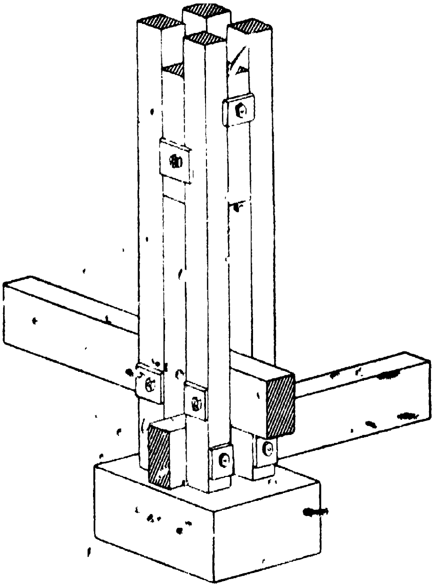

FIG. 10.

of timber of such dimensions as to fit exactly into the space between the four upright timbers must be inserted at distances of 2 or 3 feet from one another, and fixed with iron wire or bolts. This arrangement prevents the pillar from bending or twisting.

It is advisable that the foundation sills and the foot-braces should, at the four corners of the building, be crossed with a brace and fixed by bolts, as in figs. 11 and 12.

In all cases where the foot-brace system has been adopted, one or two horizontal through-braces should be applied to the pillars at different heights; they have to be fixed to the pillars by bolts, as in figs. 13 to 18, and blocks of wood should be inserted on either side of the through-braces, between the latter and the pillars.

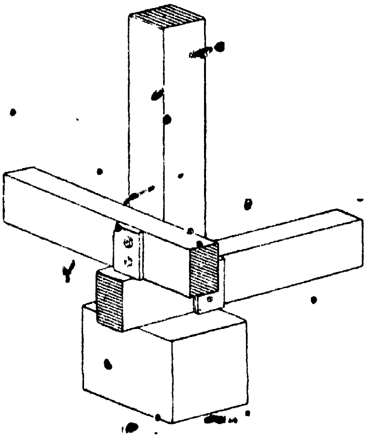
FIG. 11.

In the spaces between one pillar and the other, where there are neither doors nor windows, and where the outside appearance is of no importance, it is advisable to place struts and fasten the same to the through-braces and pillars with bolts, in order to avoid notches in the pillars, which would be a source of weakness in them.

The horizontal beams used in the framework of the building should be fixed to the pillars by means of bolts, and at the four outer corners (which are weak points of the construction) they should be united among each other by L-shaped metal

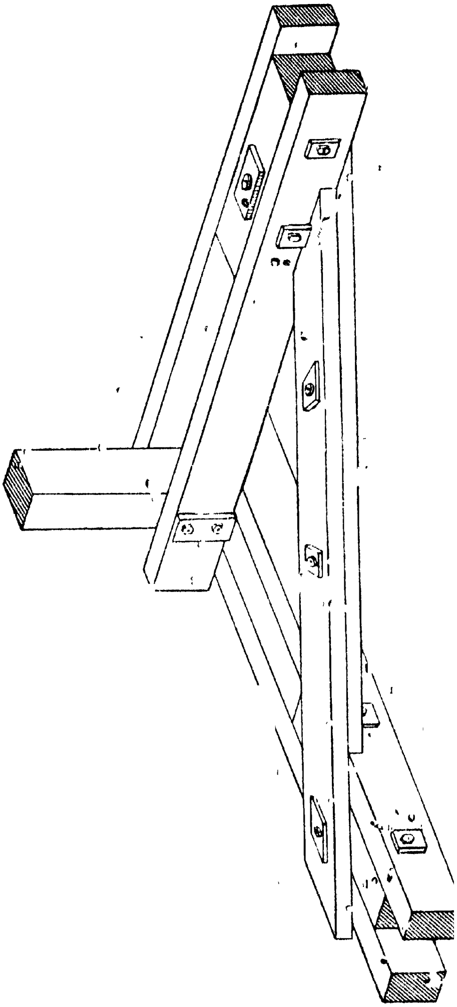

FIG. 12.

straps. In places where the outside appearance of the building is of no importance, these beams are also bound to each other

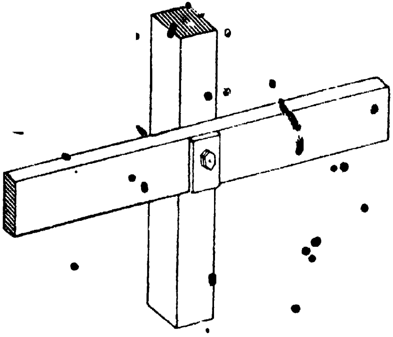

Fig. 13.

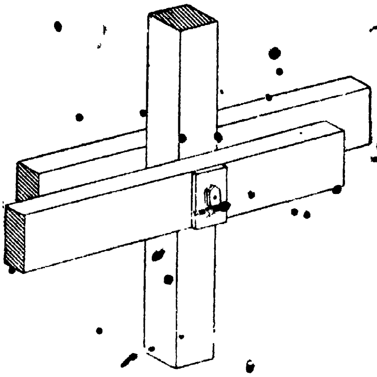

Fig. 14.

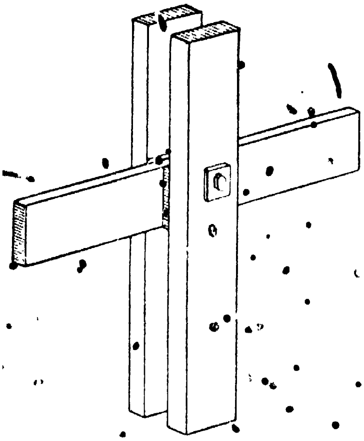

Fig. 15.

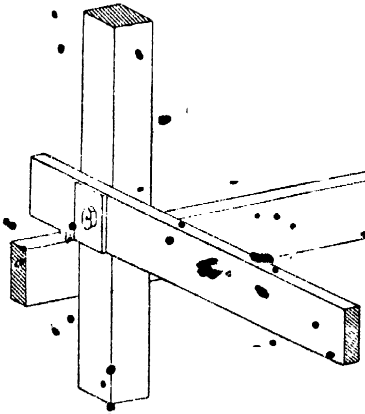

Fig. 16.

by making their extremities overlap and bolting them to the pillars.

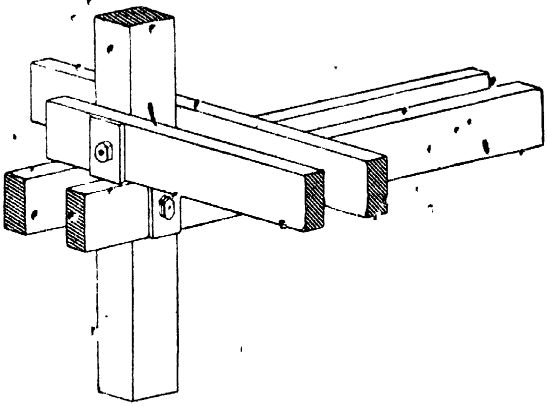

FIG. 17.

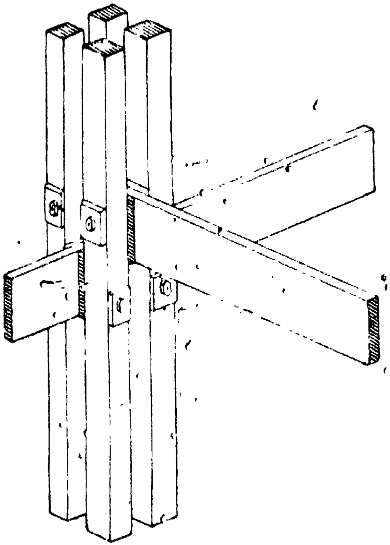

FIG. 18.

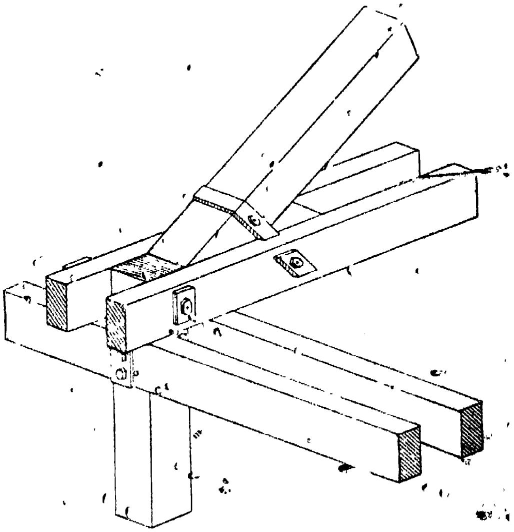

FIG. 19.

The junction of the top ties with the pillars is made similarly to that of the pillars with their foot-braces. For the junction of the roof to the walls, it is advisable to make it as in figs. 19, 20, 21, either by holding the upper part of a pillar between double tie-beams placed on wall-plates, or by using double rafters and letting tie-beams fall upon the tenon of a pillar. In either case, the junctions are made with bolts or iron straps.

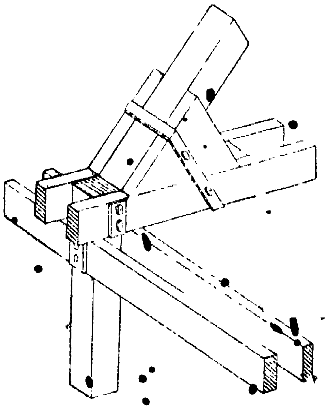

FIG. 20.

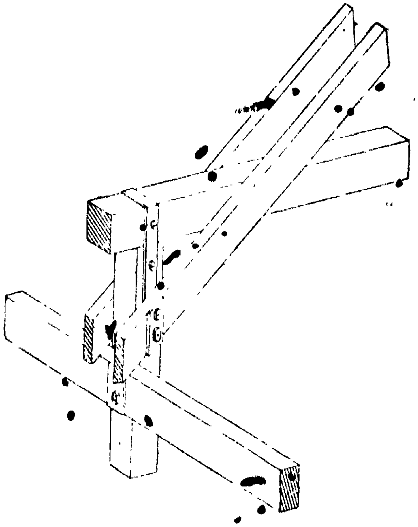

FIG. 21.

## CONSTRUCTION OF THE ROOF-FRAME.

This is done in accordance with figs. 19 to 24. The use of materials of too large dimensions should be avoided; the scantlings ought to have just the size required to support the weight of the roof itself with the pressure of the wind and the burden of the

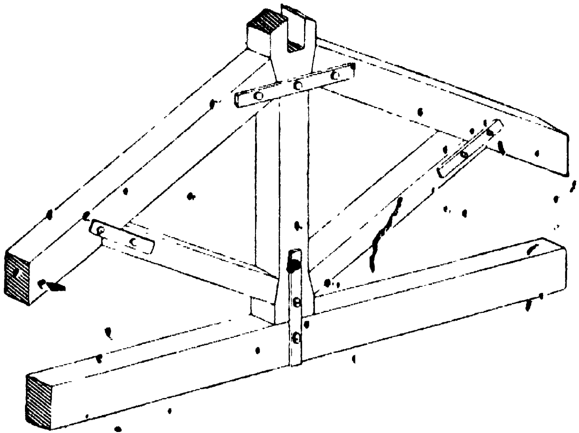

FIG. 22.

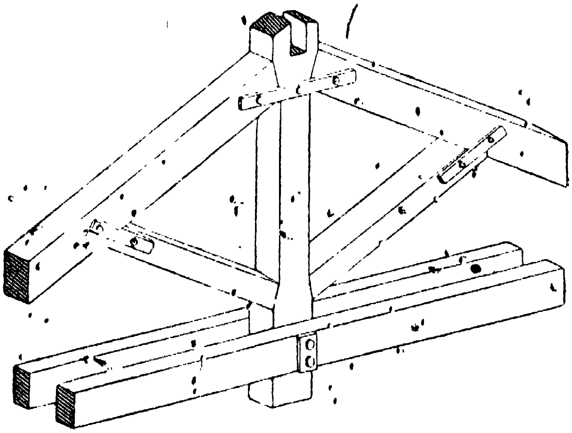

FIG. 23.

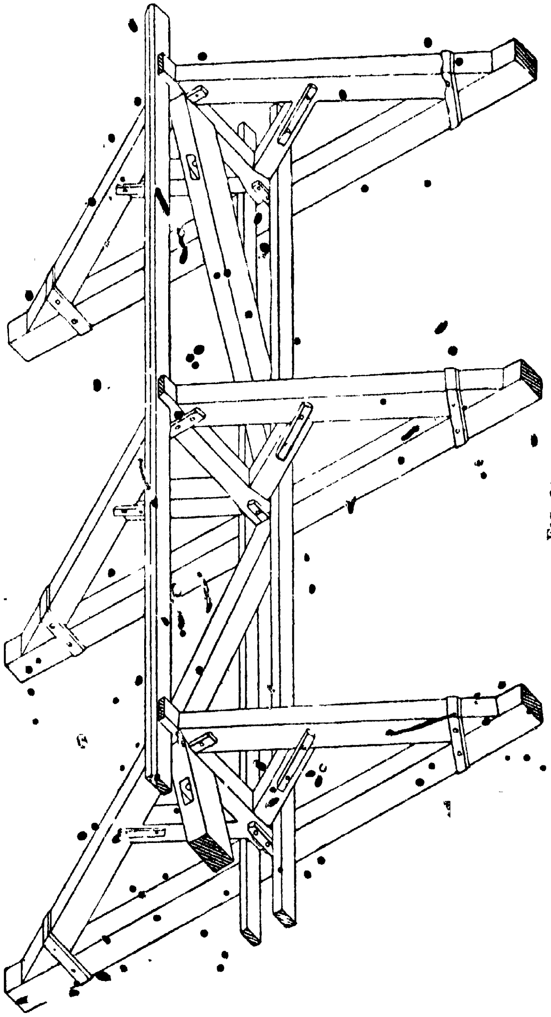

FIG. 24.

show added to it. All connections are to be made with strong iron wire, iron straps or bolts.

Between the principal rafters struts or braces, or both, should be used, and the whole framework of the roof should be bound, as shown in fig. 24, with iron clamps or bolts.

In order to enable the roof to withstand earthquake shocks, it is preferable that it should be light. It has therefore to be made as light as possible, provided it fulfils its purpose of protecting against wind and cold. If tiles are used, it will be well to attach them with nails or wire.

## JOINTS.

The joints in all parts of the building must be as simple as possible, for a complicated joint, even though it may appear strong, is really always weak. It is advisable to make the joints as in figs. 25, 26, 27, wood or iron fish-plates being fastened with bolts. In making a tenon also, the simpler its form the better.

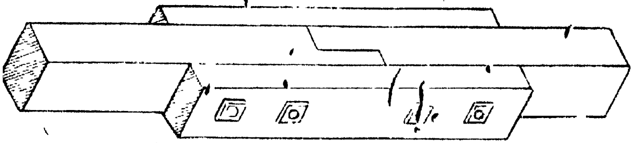

FIG. 25.

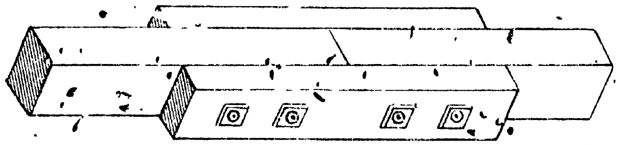

FIG. 26.

In building a house of two or three stories, it is sometimes necessary to use, instead of pillars of a single piece, a jointed pillar. In such a case the joints should be two or three feet above or below the level of the upper story of the house.

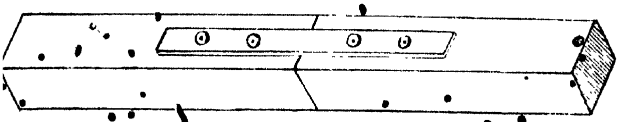

FIG. 27.

Foundation sills, foot-braces, through-braces, floor-beams, tie-beams, etc., should be fixed so as to have their outer extremities somewhat protruding.

## JUNCTION OF A PORCH OR A SHED TO THE MAIN BUILDING.

The old method of joining by tenoning or nailing has to be altogether avoided, on account of its being dangerously defective. The two buildings have to be bound together in the same manner as indicated for joining the top ties to the pillars.

## MATERIALS.

Quality and dimensions of the iron are of great importance in building, and, consequently, great attention must be paid to the quality of material as well as to the number and the distribution of the bolts. As to the washers, it is preferable that they should be large.

Timber is liable to shrinking—a reason why bolts lose their tightness. Well-seasoned timber should therefore be used.

## CONCLUSION.

The essential points in the construction of an earthquake-proof wooden building consist in the mode of making the foundation; in preserving intact in every beam its power of resistance as well as its proper functions, and if, nevertheless, some weakening of the beam should be unavoidable, in remedying the defect by the application of iron (which is much stronger than wood); in the use, whenever it is possible, of constructions of triangular form, in accordance with the principle that a triangle has an unalterable form; and finally, in reinforcing the whole framework with iron in such a manner that all the parts are bound together in one stable building.

# CHAPTER V.

# JAPANESE EXPERIMENTS AND INVESTIGATIONS REGARDING THE RESISTANCE OF BRICK COLUMNS.

Effect of the Horizontal Movement—Short Columns—Maximum Acceleration—Stability—How the Force is understood to act Materials tested—Their Resistance—Static and Dynamic Tests—Conclusions.

As mentioned in Chapter I., Omori observed that in earthquakes the destruction of buildings may be considered due to the horizontal component of the movement. The vertical one is generally much smaller, and, moreover, the examination of damaged houses shows that the vertical movement, even though it be very great, cannot produce very serious damage, except, of course, in cases where the foundations of buildings subside. According to Omori, the walls of houses of one or two stories are liable to break especially under the roof, owing to the disaccord between the horizontal vibrations of the walls and those of the roof. On this account, and also for the sake of simplicity, it will be admissible in these investigations to take account only of the horizontal component of the movement.

Omori in his paper,[1] which we here partly take up again, treats the resistance to fracturing of *short* columns, *i.e.* of those whose height is not *infinitely* great in proportion to their thickness and to the amplitude of the seismic movement, which, in the great earthquakes, may be assumed to vary between 50 and 200 mm.

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[1] F. Omori, *Publications*, No. 4, p. 69. See also the same author, *Note on Applied Seismology*, Part I., section iv. (from the Transactions of the first International Seismologic Conference, Strasburg, 1901).

Before all, it will be necessary to know how the force produced by an earthquake and acting upon a building must be understood to be applied, *i.e.* whether suddenly and repeated, or gradually. For that purpose the relation between the force and its effect ought to be considered first.

It is well known that a sudden and repeated force produces on an elastic body twice as great an effect as would have been the case if it had been applied gradually. That can be explained in the following manner.

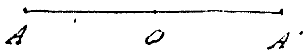

FIG. 28.

Let us assume that *A* represents diagrammatically (fig. 28) the position of equilibrium of a given body in its natural state, and *O* the position assumed by it when the force *K* is gradually acting upon it.

If the force *K* acted suddenly and repeatedly upon the body, the result would be the same as if the body whose position of equilibrium is in *O* had been displaced to *A*. As a consequence, the body would be carried into the position *A'*, which is placed at an equal distance and at the opposite side with regard to the position of equilibrium *O*, and in that manner oscillations would occur. (In other words, the strain produced by the sudden and repeated application of the force *K* is twice as great as if it had been produced by its gradual application.)

When, therefore, the force is applied gradually to an elastic body, the latter assumes its position of equilibrium without having been set into vibration.

If, however, the application of the force takes place within a time which is infinitely short in proportion to the proper vibration period of the body, the force is applied suddenly.

In the case of an earthquake the horizontal force must be understood, as regards the columns in question, to be applied gradually, for reasons that we shall see later.

By means of a shaking table whose amplitude and duration of oscillation could be varied according to the wish of the experimenter, and upon which the column to be tested was placed vertically and fixed at its base, Omori determined the strength of flexure of brick columns, the shearing stress produced by the motion of the table being negligible owing to the dimensions of the columns.[1] The height of these columns was several times, but, as we have said, not *infinitely*, greater than their thickness or the amplitude of the oscillating movement of the table. He made the table move in such a manner as to apply to the column a maximum acceleration $A$, in consequence of which the column was broken.

Comparing the value $A$, thus determined, with the value $a$ of the fracturing maximum acceleration deduced mathematically on the basis of the strength of the columns (having, of course, obtained the value of the strength by way of experiment, the breaking force being applied gradually), which expresses, therefore, the degree of stability of the column under the action of a static force, Omori found the results sufficiently in agreement with one another and of such a nature as to give also practical value to the theoretical formulæ which gave the stability of the column.

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[1] It should be noted that, from experiments made by Omori himself (see *Notes on Applied Seismology of the first International Conference, etc.*, p. 375) and by others, as, for instance, by the Austrian Commission on Vaults (see "Bericht des Gewoelbe Ausschusses," *Zeitschrift des österreichischen Ingenieur- und Architekten Vereins*, Vien. [illegible], 1895), the arsenal of Watertown, U.S.A. (see *Engineering Record*, 23rd February [illegible]), etc., it appears that brick masonry behaves almost up to breaking point like an elastic solid, and follows with sufficient approximation the law of proportionality between strains and the corresponding stresses.

The force in these experiments must be understood to have been applied gradually and not impulsively, although the motion was very violent (the period of the oscillating table in the experiments varied from 0·23 to 0·89 second, and then the acceleration of the movement passed from zero to the maximum in a time of from 0·06 to 0·22 second). That was owing to the very short natural vibration period of the column.

With much better reason it may be said that the force is gradually applied when a real destructive earthquake is in question, the period of whose principal vibration is probably one or two seconds, and therefore longer than that of the oscillating table.

It must, therefore, be understood that the effect of an earthquake upon one of these columns would be to apply to its centre of gravity a static force capable of giving to the column the maximum acceleration $A$, or else, if the column breaks, and the fracture does not occur properly at the base of it, to apply to the centre of gravity of the upper part of the broken column a force capable of giving to that part of the column the same maximum acceleration $A$.

By means of suitable instruments Omori determined the tensile strength of the brickwork of the columns he had experimented upon on the oscillating table.

The following are a few important details concerning the materials used and their strength :—

Omori used very good mortar composed of one part cement and two parts sand; he found that the tensile strength of such mortar was equal to that of bricks of rather inferior quality, and, in fact, columns of such bricks were broken, by bending, partly across the mortar and partly across the bricks. Very rarely was the fracture caused by the separation of the mortar from the bricks.

However, the tensile strength of the mortar increased when it was used with bricks of superior quality.

In all these experiments the force which caused the breaking of the material was applied very gradually. For forces applied suddenly, the strength would have been reduced to one-half.

In fact, as we have seen, the effect of a suddenly applied force is twice as great as if the force had been applied statically. That is true regarding all elastic bodies within the limit of perfect elasticity, and as regards bricks and stones it will be true up to fracture, for with these materials a rupture will occur as soon as the limit of elasticity has been overstepped.

The weight of the brickwork used was generally 0·0603 pound per cubic inch (equal to 1670 kgs. per cubic metre).

Tests made, to give an instance, with the material of a column nine months after its construction gave the following results :—

Tensile strength of the bricks, 142 English pounds per square inch (1 pound = 0·454 kg.; 1 inch = 25·4 mm.).

Tensile strength at the joint of the brick with the mortar, 71 pounds per square inch.

Among four tests, the mean result of which was 71 lb., are two in which the separation took place clean at the joints. The mean strength for these two was 37·6 lb., which must be regarded as the lowest strength of the column.

The bricks in question were of a quality known as extra second class.

The force was applied gradually at intervals of from one to three minutes.

The tensile strength of the bricks alone was found to vary for extra superior quality (all of the same quality) between 227 and 365 lb. per square inch, mean strength 303 lb., for a very gradually applied force. It was also proved, in applying the weight rather violently, that its application took place in less than $\frac{1}{10}$ of a second. When in that manner once 227 lb. and another time 255 lb. per square inch were applied to one brick, it did not break. Later on it broke under a very gradual application of 356 lb.

In a similar manner dynamic tests were also made of 189 and 217 lb. upon another brick without any rupture occurring, but rupture did take place at the static test with 257 lb.

That denotes that the action was not yet even then a sudden one, the rupture at the dynamic test not having occurred though the tension applied was much greater than one-half of that which had produced the rupture at the static test.

In these experiments the tensile strength of the brickwork varied from 33·7 to 130·4 lb. per square inch. This great difference resulted perhaps to a certain extent from the quality of the bricks, for all the columns were made with the same mortar. It seems, as we have seen, that the tensile strength of bricks and mortar taken together increases with the good quality of the bricks.

It appears that this strength depends but little on the dimensions of the bricks or on their distribution in the brickwork, *i.e.* whether that be made with horizontal joints only, or with horizontal and vertical joints.

The strength seems to vary a great deal with the more or less accurate workmanship. In one case the mortar was found to adhere to the whole surface of the brick, in a second case to half of it, and in a third only in some detached points. The tensile strength was 209, 119, and 61 lb. respectively per square inch. Very careful supervision is, therefore, required in the execution of the work.[1]

Omori also gives, in the paper from which we have culled the above details, the theoretical formulæ of the profiles (parabolic) of brick columns of various sections of equal strength of flexure, assuming that they are of perfectly elastic material. These formulæ are also applicable to walls and bridge piers.

In the following chapters we give, with some deductions, the calculations of columns and walls of bricks and of reinforced concrete. We have made these calculations starting from well-known formulæ of building science, and following the ideas of Omori on the action of the seismic force. By another way we come back again to the formulæ already given by Omori of the parabolic profile for brick columns.

---

[1] The Austrian Commission on Vaults (*loc. cit.*) found that vaults built of machine-made, sufficiently resistant bricks with mortar of slowly hardening Portland cement (1 part cement and 2·6 parts sand), age of the masonry about 1½ months, had a tensile strength from 4 to 6 kgs. per sq. cm.

# CHAPTER VI.

## CALCULATION OF BRICK COLUMNS & WALLS OF UNIFORM STRENGTH BY BENDING.

Prismatic Columns—Maximum Bending Moment—Section of Rupture—Calculation of the Stability—Parabolic, Rectangular, and Square Columns—Walls—Weight-bearing Walls—Application to the Seismic Forces—Numerical Example—Verification of Stability.

LET us assume that $ABCD$ (fig. 29) is a prismatic column, elastic, homogeneous, erected vertically on level ground and imbedded to the section $EF$. Let this column be subjected along its free part to horizontal forces, distributed uniformly along its axis, applied gradually, and capable of giving to the corresponding elements of the column the acceleration $a$.

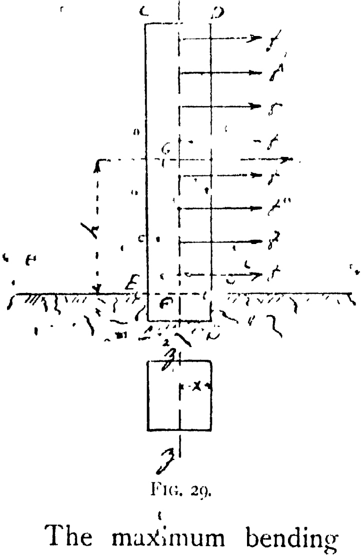

FIG. 29.

Let $f$ be the value of these forces per unit of the length of the body; $m$, the mass of the body, also per unit of length. We then have $f = ma$.

The maximum bending moment $M$ to which the column is subjected on the part of the forces $f$ will evidently be in correspondence with the section $EF$ where the column is fixed down.

Let $F$ be the resultant of the forces $f$, and $P$ the weight of the column $ABCD$, $g$ the acceleration due to gravity, and we shall have

$$F = \frac{P}{g} a ,$$

$\frac{P}{g}$ being the mass of $CDEF$. When there is a question of finding the bending moment $M$ in correspondence with the section $EF$, it is admissible to substitute for the $f$'s the force $F$ resulting from them. That force must be understood to be applied in $G$, the centre of gravity of $CDEF$. If, then, $h$ signifies the height of $G$ above the section $EF$, $h$ is equal to one-half the height of $CDEF$. We have

$$M = Fh = \frac{P}{g} ah .$$

Let us take $I$ as the moment of inertia of the section of the solid, $x$ the distance of the farthest distant fibre from the neutral axis $zz$, $R$ the ultimate tensile or compressive stress (the smallest of the two) of the material of which the body is made—it is known that the maximum bending moment of which the solid is capable before it breaks is equal to

$$M = R \frac{I}{x} ;$$

if it is admitted, as we have done, that the law of proportionality between the strains and the stresses manifests itself up to the rupture point, and if the influence of the column's own weight is neglected in the computation of $R$.

The force $F$ or the acceleration $a$ which are capable of causing the rupture of the column are therefore given by

$$\frac{P}{g} \cdot ah = R\frac{I}{x}$$

$$a = \frac{RIg}{x.Ph}.$$

If we call $p$ the weight of the matter per unit of volume, and $V$ the volume corresponding with $P$, we have $P = pV$, and, therefore,

$$a = \frac{RIg}{pVhx} \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

This is, therefore, the formula which gives $a$ capable of breaking by flexure a prismatic column imbedded at the base, for those elastic solids for which the law of proportionality is admissible up to rupture. Approximately this formula can also be applied to other forms in which the variation of section takes place very gradually.

Let us make out the following particular case:—

*A prismatic column of rectangular section*, of sides $2x$ and $b$, $b$ being the side parallel to the neutral axis.

Then there is $I = \frac{2}{3}bx^3$, $V = 4hxb$, and we have from formula (1)

$$a = \frac{2xgR}{3p(2h)^2} \qquad (2)$$

In this formula the dimension $b$ does not figure, and it can assume different values. Assuming $b = 2x$, we have a column of square section; assuming $b$ to be very great, we have a wall of constant thickness in its entire height, and subject to stresses normally in its length.

As we have said, our prismatic solid must, at least theoretically, break by flexure at its base, because the maximum bending moment is there. Let us now look for the profile of a column which offers the same resistance to rupture by flexure in every section of it. We can state at once that it will have a more massive form at the bottom and less at the top, and that the variations of dimension occur gradually.

Let us call $S$ the area of the base of the column and $V$ its volume, and we shall have $V = Sky$, $y$ being the height, and $k$ a numerical coefficient smaller than 1. Let us assume, for instance, the section of the solid to be rectangular, of sides $2x$ normal and $b$ parallel to the neutral axis. There will then be $S = 2xb$, $V = 2xbky$. Calling $h$ the height of the column's centre of gravity above the base or the section at level of the soil, the end of the column being imbedded in the ground, we can also write $h = k'y$, where $k'$ is a coefficient smaller than 1. Taking $I = \frac{2}{3}bx^3$, we have from formula (1)

$$\alpha = \frac{1}{3}\frac{gRx}{kk'py^2},$$

that is,

$$y^2 = \frac{gR}{3kk'p\alpha}x.$$

This is the equation of a parabola of which $O$ is the vertex, $Ox$ the axis of symmetry, $Oy$ the tangent in the vertex, and which is related to these axes $Ox$ and $Oy$. The geometrical locus of the points $P$ which constitute the extremes, on one side of the symmetry axis $Ox$ of the solid, of the sides of the horizontal sections normal to the neutral axis, is given therefore by a parabola, for instance $OB$ of fig. 30. This shows, therefore, that the column has a parabolical form.

After these premisses, let us try to determine the column.

Let $b$ be $=mx$, $m$ being a constant numerical coefficient for the various horizontal sections. For every section the $h$ and the $V$ of formula (1) have to be determined.

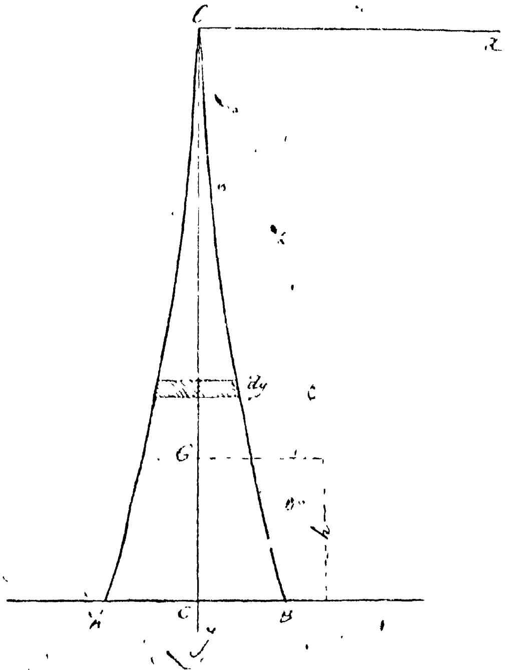

FIG. 30.

*Determination of h.*—What has to be found is the centre of gravity $G$ of a right, rectangular, parabolic pyramid $BOA$ with regard to the axis $x$ which passes through its vertex $O$, normal to $Oy$ axis and parallel to the side $2x$ of every horizontal section.

Divide the pyramid into so many trapezoids, each of ~~them of~~ the volume $2xbdy$. The moment of one of them with regard to the axis $x$ is equal to $2xbdy.y$. For the pyramid of base $BA = 2x$ and height $OC = y$ there is

$$\int_o^y 2xbdy\,.\,y = OG\int_o^y 2xbdy\,.$$

Calling $q$ the parameter of the parabola, we have

$$y^2 = qx, \qquad x = \frac{y^2}{q}, \qquad q = \frac{y^2}{x}.$$

and therefore,

$$\int_o^y \frac{y^5}{q^2}dy = OG\int_o^y \frac{y^4}{q^2}dy,$$

$$OG = \frac{5}{6}y, \qquad h = CG = \frac{y}{6}.$$

*Determination of V.*—$V = \int_o^y 2xbdy = 2m\int_o^y x^2dy\,.$

Carrying out the calculation, and bearing in mind that $y^2 = qx$, we have

$$V = \frac{2}{5}mx^2y$$

Substituting now in formula (1), as $I = \frac{2}{3}bx^3 = \frac{2}{3}mx^4$, we have

$$a = \frac{2}{3}\,\frac{mx^4gR}{x\frac{y}{6}p\frac{2}{5}x^2ym} = 10\frac{xgR}{py^2},$$

$$y^2 = 10\frac{xgR}{pa} \qquad . \quad . \quad . \quad . \quad . \quad (3)$$

The numerical coefficient $m$ is arbitrary, and, moreover, it does not figure in formula (3). Assuming $m = 2$, we have the square section, to which, therefore, the formula (3) is applicable.

Let us select another important case, viz.: let $b$ be = constant. The column is reduced to a line of wall of constant length $b$, and of parabolic profile.

*Determination of h.*—Using the foregoing data, we have

$$\int_o^y 2xbdy\,.\,y = OG\int_o^y 2xbdy,$$

and bearing in mind that $y^2 = qx$, we obtain

$$OG = \frac{3y}{4}, \qquad h = \frac{y}{4}.$$

*Determination of V.*—We have

$$V = \int_o^y 2xbdy,$$

and again bearing in mind that $y^2 = qx$,

$$V = \frac{2}{3}xby.$$

Substituting these expressions in formula (1), as $I = \frac{2}{3}bx^3$, we have

$$a = \frac{2}{3}\frac{bx^3gR}{\left(x\frac{y}{4}\right)\left(p\frac{2}{3}xby\right)} = 4\frac{gRx}{py^2},$$

$$y^2 = \frac{4gR}{pa}x \qquad . \quad . \quad . \quad . \quad (4)$$

In analogous manner the profiles of columns of any section whatever can be determined. It will be sufficient to introduce some slight variation into these calculations which we have set out in a detailed manner. In continuing, we shall speak only of walls: what we shall say of them can with the greatest facility be applied to columns in general.

Supposing we are to construct a parabolical wall of height $H$, and destined to support for every portion $b$ of its length a uniformly distributed weight $P'$, for instance, a roof. Let us assume that the wall in question is a brick wall.

Let $CD$ (fig. 31) be the base to be calculated of this wall, and

$M$ the bending moment produced at the base of the wall by the mass of the roof, to which the acceleration $a$ has been given. We then have

$$M_1 = \frac{P'}{g} a \, . \, H.$$

Now, let us imagine that the parabolical wall $ABCD$ which is

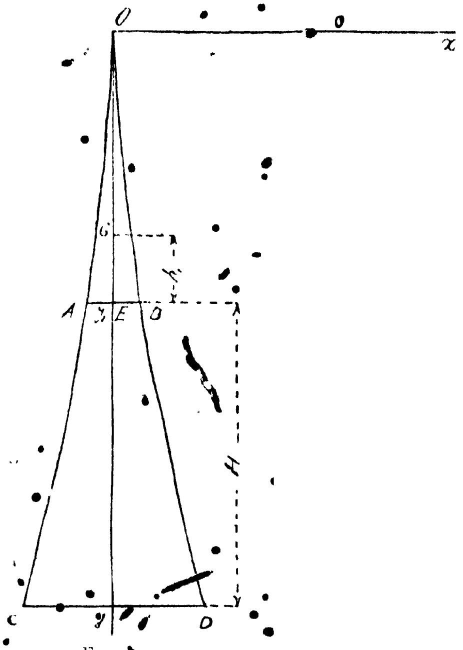

FIG. 3.

to be erected has been continued up to the point $O$ of its profile. If we call $V_1$ the volume of the part of the wall above the section $AB$, and supposing the length of the wall to be equal to $b$, we get:

$$V_1 = \frac{2}{3} x_1 y_1 b,$$

being $AB = 2x_1$, $OE = y_1$.

Calling $P_1$ the weight corresponding with $V_1$, we have $P_1 = pV_1$.

Let $h_1$ be $= \frac{y_1}{4}$, the height of the centre of gravity $G$ of $AOB$ above $AB$.

The roof, of the weight $P'$, is put upon $AB$. The weight $P_1$ of the portion $AOB$ of the wall is applied in $G$, and that means it is distant from $CD$ by $H + h_1 = H + \frac{y_1}{4}$.

Let us now give to the portion $AQB$ of the wall that height, *i.e.* let us make $AB$ so great that we have

$$\frac{P'}{g}aH = P_1\frac{a}{g}\left(H + \frac{y_1}{4}\right) = pV_1\frac{a}{g}\left(H + \frac{y_1}{4}\right).$$

The base $CD$ of the wall, the latter being calculated as continued up to $O$ and not bearing any weight, will be the same as for the wall truncated at the height $H$ and burdened with the weight $P'$ of the roof. The other horizontal sections of the wall, calculated as if the latter were continued to $O$, will offer an excess of resistance in case the wall should be truncated along $AB$ and burdened with $P'$. Let us, therefore, calculate the wall as if it were unburdened and continued to $O$, and being capable of satisfying the last equation.

Let, therefore,

$$P'H = pV_1\left(H + \frac{y_1}{4}\right) = \frac{2}{3}pbx_1y_1\left(\mathrm{H} + \frac{y_1}{4}\right),$$

that is,

$$P'H = \frac{2}{3}pHbx_1y_1 + \frac{1}{6}pbx_1y_1^2 \quad . \quad . \quad . \quad . \quad . \quad (5)$$

By formula (5) and by

$$y_1^2 = \frac{4gR}{pa} \cdot x_1 \quad . \quad . \quad . \quad . \quad . \quad (6)$$

(which is only formula (4) applied to section $AB$), we determine $x_1$ and $y_1$, *i.e.* section $AB$ itself.

*Numerical Example.*—Supposing $P' = 1000$ kgs., $b = 100$ cm., $H = 700$ cm. Let us assume as the weight of the brickwork 1600 kgs. per cubic metre, *i.e.* let $p$ be $= 16 . 10^{-4}$ kg./cub. cm. Bearing in mind the results of Omori's experiments, let us assume $R = 3$ kgs. per sq. cm., which corresponds with 40 lb. per square inch. To be more exact, $R$ ought to be increased by the weight per unit of area of the section of rupture of that part of the wall which is above that section, the wall having to be considered as being extended up to $O$. Generally, however, in computing $R$ this weight may be neglected, the more so as in that case a safety margin is obtained. We shall later on make allowance for it in the calculation of verification. Let us choose a coefficient of stability $a = 4000$ mm. *i.e.* $= 400$ cm.

From formulæ (5) and (6) we have

$$1000 . 700 = \frac{2}{3} 16 . 10^{-4} . 700 . 100 x_1 y_1 + \frac{1}{6} 16 . 10^{-4} . 10^2 x_1 y_1^2 \quad . \quad . \quad (a)$$

$$y_1^2 = \frac{4 \cdot 981 . 3}{16 . 10^{-4} . 400} x_1 = 184 . 10^2 x_1 \quad . \quad . \quad . \quad . \quad (b)$$

From $(b)$ we have

$$y_1 = 136 \sqrt{x_1} .$$

And substituting in $(a)$,

$$7 . 10^5 = \frac{2}{3} . 16 . 7 . 136 x_1^{\frac{3}{2}} + \frac{1}{6} . 16 . 184 . x_1^2 .$$

That is to say, we have an equation of the form $a = bx^{\frac{3}{2}} + cx^2$ which can be easily solved by finding the value of $x$.

We find that $x_1 = 15$ will satisfy the equation. Substituting that value in $(b)$, we have $y_1 = 520$ cm. For the base $CD$ we have, therefore,

$$y_1 = 520 + 700 = 1220 ,$$

and substituting that in $(b)$ we obtain $x = 80$.

The wall has therefore a width of 30 cm. at the top and of 160 cm. at the base. In practice, the sides $AB'$ and $CD$ of its vertical section will be united by straight lines, as in fig. 32, instead of by arcs of parabola. That does not affect the stability, while it simplifies the construction.

Calling $P$ the weight of the wall per lineal metre, we have

$$P = \frac{160+30}{2} . 700 . 100 . 16 . 10^{-4} = 10{,}600 \text{ kgs.},$$

and it will be applied to a height $h$ above the base given by

$$h = \frac{160+2 . 30}{160+30} . \frac{700}{3} = 270,$$

which we have obtained from a known geometrical formula.

The acceleration $a$ of which we have spoken up to now may be understood to be applied to the column or wall in the following manner :—

Let us assume that a seismic force imparts to the soil an oscillating movement whose maximum acceleration is $a$. If we imagine that simultaneously there should have been communicated to the columns and the soil an oscillating movement identical with that which the soil has really received and in the same direction, but of phase shifted to 180°, then the soil will remain immovable, and a horizontal force $F$ may be understood to have been applied to the column. If, in accordance with what we have seen in the preceding chapter, the height of the column be not *infinitely* great compared with its thickness and the amplitude of the seismic movement, the force $F$ of the earthquake may be understood as having been applied gradually to the centre of gravity of the column. Consequently the formulæ which we have deduced in the present chapter are applicable to the construction of columns and walls in seismic regions, but of course only in the case of short columns. The maximum acceleration of 4000 mm. per sec. per sec. which we have assumed as coefficient of stability in our numerical example represents in practice a very considerable value, and, generally speaking, it will not be required to base the calculation of construction upon higher values in seismic regions.

Once a masonry column has been calculated according to the preceding formulæ, it might be advisable for the builder to proceed in the usual manner to the verification of stability. It is known that the conditions of stability required in the practical building of a wall are as follows :—

1. Resistance to rotation in every horizontal joint.
2. Resistance to sliding stress in every horizontal joint.
3. Resistance to crushing stress in every horizontal joint.
4. In order to prevent the mortar being affected by tension, the resultant of the forces must for every horizontal joint pass within the middle third of the base.

*Resistance to Rotation.*—Seeing that we base our calculations also upon the tensile strength of the mortar, and that we consider the wall to be an elastic solid imbedded at its base, rotation is not possible. Incidentally we make the following remarks.

According to Omori, when the columns in question are small and simply put with their bases on the soil, so that neither the period of their rocking before they are overturned will be long in comparison with the period of the earth motion, nor their thickness great in comparison with the amplitude of the telluric movement, the arguments set down before regarding short columns subjected to flexure may be repeated here, and the seismic force may be understood as being applied gradually to the centre of gravity of the column itself. If we then call $x$ the semi-thickness of the column, and $h$ the height of its centre of gravity above the base, it is necessary for overturning the column that the turning moment of the horizontal force about the edge of the basis should be greater than the moment of the weight about the same point, *i.e.* there must be

$$a > \frac{x}{h} g,$$

a formula due to Professor C. D. West.

As regards very large columns, the seismic force must, on the other hand, be understood as being applied suddenly. In that case the overturning will in general become practically impossible, because the amplitude of the earthquake motion will not be sufficiently great.

*Resistance to Sliding Stress.*—Assuming $\frac{a}{g} = \tan \delta$, there is in our numerical example $\frac{a}{g} = 0{\cdot}41$, and therefore $\delta = 22^\circ 20'$, which means, the resultant of the forces applied to every joint forms with the vertical an angle of $22^\circ 20'$. In order to prevent by friction the horizontal joints in a wall sliding over each other, there must be $\delta < 35^\circ$. Here there would, therefore, be no sliding even if we do not take the resistance of the mortar into account.[1]

*Resistance to Crushing.*—Let us consider an elastic, prismatic solid whose every transversal section presents a symmetry axis normal to the line of intersection of the plane of the section with the plane containing the resultant force acting on the solid, and whose fibres oppose equal resistance to tension and compression. The neutral axis will then coincide in every section with the axis of symmetry, and the farthest compressed fibre as well as the farthest stretched fibre will be at equal distances from the neutral axis. Let us call $A$ the area of the transversal section of the solid, $P$ the force along the axis of that section, $M$ the bending moment at the section, $I$ its moment of inertia, $z$ the distance of the farthest fibre from the neutral axis. As is well known, the maximum unitary compressive stress $\sigma_1$ and tensile stress $\sigma_2$ sustained by the farthest distant fibres of the section are:

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[1] See also the note on p. 6, Chapter I., relating to the non-horizontal acceleration of the earthquake.

$$\sigma_1 = \frac{P}{A} + \frac{M}{\frac{I}{z}}; \qquad \sigma_2 = \frac{P}{A} - \frac{M}{\frac{I}{z}} \quad . \quad . \quad . \quad . \quad (7)$$

These formulæ will approximately stand also in our case of a solid with a profile slightly inclined on the axis of the solid itself, and not perfectly elastic. When we have to deal with a rectangular section of sides $2x$ parallel and $b$ normal to the force of the earthquake, these formulæ are translated into the following:

$$\sigma_1 = \frac{P}{2x\,b} + \frac{Pd}{\frac{1}{6}\,.\,4x^2b} = \frac{P}{2x\,.\,b}\left(1 + \frac{6d}{2x}\right)$$

$$\sigma_2 = \frac{P}{2x\,.\,b}\left(1 - \frac{6d}{2x}\right),$$

in which

$$d = \frac{M}{P},$$

In our case, if we call $ABCD$ (fig. 32) the profile of the wall, the weight $P'$ is applied in $E$, and the own weight P″ in $G$. The total weight $P$ is applied in $R$ in such a manner that $\frac{P'}{P''} = \frac{GR}{RE}$. $R$ rises 37 cm. above $G$, or 307 cm. above $AB$.

Every metre-run of the wall is, therefore, under the action of a vertical force $P = 11{,}600$ kgs., and of a horizontal force F $= \frac{a}{g} 11{,}600 = 4750$ kgs. applied in $R$. Our case is (fig. 32):

$$d = ST = \overline{RS}\ \text{tg}\ \delta = 0{\cdot}41\,.\,307 = 125.$$

We have, therefore,

$$\sigma_1 = \frac{11,600}{2.80\ 100}\left(1 + \frac{6.125}{2.80}\right) = 4{\cdot}1[illegible]$$

$$\sigma_2 = -2{\cdot}68.$$

The compressive strength of the masonry depends upon the quality of the bricks, the mortar, the age and the form of the solid, but in any case it is vastly superior to the value of σ as found here. Our wall can therefore very well resist crushing.

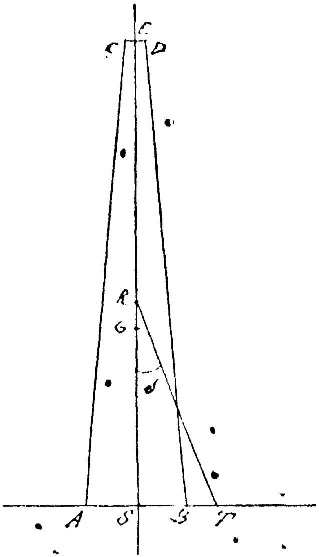

FIG. 32.

*Tensile Strength of the Mortar.*—From the above it results that our brickwork is subject to a maximum tensile stress of 2·68 kgs. per sq. cm. We have seen that the tensile strength of brickwork may be set down at 3 kgs. per sq. cm. The wall is therefore also able to resist the tensile stress.

# CHAPTER VII.

# CALCULATION OF WALLS OF REINFORCED CONCRETE.

Walls with Double Reinforcement—Walls of Little Height with Equal Strength of Flexure—Weight-bearing Walls—Period of Vibration—Impulsive Force—Centres of Rotation—Their Determination—Particular Cases—Experimental Notions—Numerical Example—Very High Walls.

We shall assume, for the purposes of this calculation, that we are dealing with a wall doubly reinforced by vertical rods of iron. Fig. 33 represents the horizontal section of a portion of a wall of length $b$; $a$, $a$ are two rods of iron destined to resist the tensile stresses which the section of the wall has to sustain. The concrete admittedly will resist compression only. For the greater simplicity of our calculation we shall reduce the section represented by fig. 33 to that shown by fig. 34, *i.e.* we shall assume that the wall is only reinforced on one side—that one on which it is subjected to tension. That is permissible in practice.

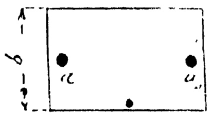

FIG. 33.

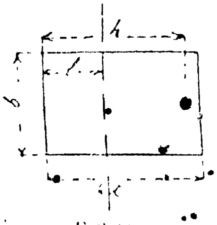

FIG. 34.

Let us call:

$M$ the bending moment acting upon the section.

$l$ the distance of the neutral axis from the farthest distant compressed fibre.

$h$ the distance of that fibre from the axis of the reinforcing rod of iron.

$F$ the section of the rod.

Supposing that the iron is strained by 1050 kgs./sq. cm., and the concrete by 35 kgs./sq. cm., strains which are admissible for concrete of the usual mixture of ingredients, but of select quality.

We then have[1]

$$\left.\begin{aligned} h &= \frac{1}{2}\sqrt{\frac{M}{b}} \\ F &= \frac{1}{240}bh \\ l &= \frac{1}{4}h \end{aligned}\right\} \qquad (8)$$

By these formulæ, and given the bending moment acting upon the section, the latter is determined. Let us now try to calculate a wall of equal resistance to flexure.

Let us call $P$ the weight of that part of the wall above the section with which we deal; $f$ the distance of the centre of gravity of the portion of the wall itself from the section; $V$ the volume corresponding with $P$; $a$ the acceleration by which the wall is to be strained; $p$ the specific weight of the reinforced concrete. We then have

$$M = \frac{P}{g}af = \frac{pV}{g}af$$

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[1] See C. Guidi, *loc. cit.*, p. 81.

## CALCULATION OF WALLS OF REINFORCED CONCRETE.

Assuming for simplicity that $h = 2x$, $2x$ being the thickness of the wall, and calling $k$ and $k'$ certain coefficients smaller than 1, and in general employing the data already used in Chapter VI., we have

$$M = \frac{a}{g} pkySk'y = \frac{a}{g} pK \, . \, 2xby^2,$$

in which $K = kk'$.

But from formula (8) results:

$$x = \frac{1}{4}\sqrt{\frac{M}{b}}; \qquad M = 16x^2b;$$

and equalising the two expressions of $M$ which we have found, we have

$$16x^2b = \frac{a}{g} pK \, . \, 2xby^2,$$

$$y^2 = 8x \frac{g}{apK}.$$

This is the equation of a parabola. Our wall will, therefore, have a parabolical profile, and from Chapter VI. it results that then there is

$$f = \frac{1}{4} y; \qquad V = \frac{2}{3} xby;$$

wherefore, proceeding in analogous manner, we have here

$$K = \frac{1}{4} \, . \, \frac{1}{3} = \frac{1}{12},$$

and consequently

$$y^2 = \frac{96g}{ap} x \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

*i.e.*, $p$ being for reinforced concrete $= 25 \, . \, 10^{-4}$,

$$y^2 = 375 \cdot 10^5 \frac{x}{a} \quad . \quad . \quad . \quad . \quad . \quad . \quad (9')$$

It must be noted that by the application of this formula it is anticipated that the reinforced concrete will be strained by the force of the earthquake not up to breaking point, but only up to a certain limit which leaves a very convenient margin for the resistance of the wall $a$ does not, therefore, represent any longer properly the degree of safety of the wall. As, however, the dimensions of the wall do not become excessive, we have considered it advisable to start from formula (8).

In case it should be considered desirable to subject, analogous to the brick walls, the reinforced concrete to a greater strain, it will always be possible to choose for $a$ a lesser value than that of the maximum acceleration $A$ anticipated for the earthquake.

A wall of a given height $H$ which bears a given weight $P'$ on its top will, analogously to what we have said in Chapter VI., be determined by

$$P'H = \frac{2}{3} pHbx_1y_1 + \frac{1}{6} pbx_1y_1^2,$$

$$y_1^2 = 375 \cdot 10^5 \frac{x_1}{a}.$$

*Numerical Example.*—Let us set down $P' = 1000$ kgs. per running metre, and $H = 7$ metres. Then we will have

$$10^3\ 7\ 10^2 = \frac{2}{3} .25 \cdot 10^{-4} . 7 . 10^2 . 10^2 . x_1y_1 + \frac{1}{6} . 25 \cdot 10^{-4} x_1y_1^2,$$

$$y_1 = 375 \cdot 10^5 \frac{x_1}{a} = 94 . 10^3 x_1.$$

Solving these two equations by attempts in an analogous manner to what we did in Chapter VI., we find

$$x_1 = 6; \qquad y_1 = 750.$$

We obtain then

$$x = 22.$$

Our wall has therefore a thickness of 12 cm. at the top and 44 at the base. In practice it would be more convenient, for the greater simplicity of the construction work, to substitute for the two parabolic arcs of the profile their spans.

The calculation which we have been pursuing will only be correct if we can assume that the force of the earthquake may be understood as being applied gradually. Let us now see whether that would be admissible in the case before us.

Omori[1] has found by way of experiment that for a column 180 cm. high and 11 cm. thick the complete vibration period $T$ was 0·2 second. From his experiments it results furthermore that, as it ought to be, the vibration period of a column varies in direct ratio to the square of its height, and in inverse ratio to its thickness. That has been verified by him for columns in solid and in hollow brickwork. As a general maxim, and also in order to give an idea of how matters stand, we shall assume that the same results can be applied to our parabolical wall of reinforced concrete, although certainly brickwork and reinforced concrete are bound to behave differently from each other.

As thickness of the wall we shall take the mean value between the thickness at the top and that at the base, *i.e.* 28 cm. We have therefore

$$T = 0{\cdot}2\,\frac{7^2 \cdot 10^4}{180^2} \cdot \frac{11}{28} = 1{\cdot}2\,.$$

Seeing that the period of a destructive earthquake may be assumed to be from 1 to 2 seconds, it cannot be said that the vibration period of the wall is very small compared with that of the earthquake. The force can, therefore, no longer be understood as being applied gradually.

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[1] See *Publications, etc.*, No. 4, p. 119.

Let us now examine what will happen when the force applied to our wall or column is sudden.

Let $AB$ be a column or a rectilinear rod in the open space, and let the impulsive force $F$ normal to the axis of the column be applied to an extreme $A$. The column will then tend to move in the plane determined by its axis and the force $F$. Let us now look for the position of the instantaneous centre of that movement or gyration. That can be done seeing that at any instant the moment of the applied force is equal to the sum of the moments of the forces of inertia of the column with regard to that instantaneous centre, and that the sum of the forces of inertia of the column's various elements is equal to the impressed force. Thus we have two equations which give us the position of the instantaneous centre of gyration which we are looking for.

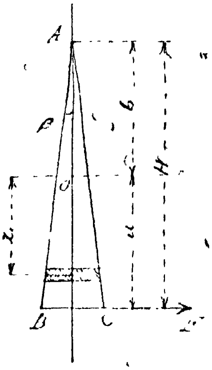

FIG. 35.

We shall make our research for some specific case. We deal with a piece of a wall of constant length and triangular profile whose vertical section is represented by fig. 35. Let us suppose that we have to do with an elastic and homogeneous solid at whose base the force $F$ (normal to the length of the wall) is suddenly applied. We shall call $O$ the centre which is to be determined, and we shall assume that the thickness of the column is negligible in view of its height.

Let us imagine the piece of wall to be divided into elements by so many horizontal sections at very short distances from each other (the distance to be called $dx$), and let us call $x$ the distance of one of those elements from the centre $O$, and $z$ the length of the piece of wall. The volume of one of those elements $mnrs$ is then approximately given by $rs.z.dx$, *i.e.* if we put $\beta = BAC$, by $2 \tan \beta(b \pm x)z\,dx$, and if we call $p$ the weight per unit of volume, and $g$ the acceleration due to gravity, the mass of one element is equal to

$$\frac{p}{g} \,.\, 2 \tan \beta(b \pm x)z\,dx;$$

and that means that if we put $2\frac{p}{g} \tan \beta \,.\, z = C$, the said mass is equal to $C(b \pm x)dx$, and we shall understand it to be concentrated on the space $dx$ of the straight line $AD$.

Call $\omega$ the angular velocity possessed by the column during the impulse in the instant $t$. The velocity $v$ of an element distant by $x$ from $O$ will then be $\omega x$. Its derivate is equal to

$$\frac{dv}{dt} = x\frac{d\omega}{dt}$$

and represents the acceleration impressed upon the element we are considering. The *vis inertiæ* corresponding with it is equal to

$$C(b \pm x)dx . x\frac{d\omega}{dt} .$$

As we have already observed, the moment of the impressed force $F$ with regard to $O$ must be equal to the sum of the moments of the forces of inertia of the various elements of the column, as there must be at every instant equilibrium between the forces applied and the forces of inertia.

The moment of one of the forces of inertia is

$$C(b \pm x)dx . x^2\frac{d\omega}{dt},$$

and consequently, if we put $C\frac{d\omega}{dt} = C_1$,

$$Fa = C_1\left[\int_0^a (b+x)x^2dx + \int_0^b (b-x)x^2dx\right],$$

and, making the integrations,

$$Fa = C_1\left(\frac{ba^3}{3} + \frac{a^4}{4} + \frac{b^4}{3} - \frac{b^4}{4}\right) = C_1\left(\frac{ba^3}{3} + \frac{a^4}{4} + \frac{b^4}{12}\right) \quad . \quad . \quad . \quad (a)$$

Equalising the forces, we have

$$F = C_1\left[\int_b^a (b+x)x\,dx - \int_a^b (b-x)x\,dx\right],$$

*i.e.*

$$F = C_1\left(\frac{ba^2}{2} + \frac{a^3}{3} - \frac{b^3}{2} + \frac{b^3}{3}\right) = C_1\left(\frac{ba^2}{2} + \frac{a^3}{3} - \frac{b^3}{6}\right) \quad . \quad . \quad . \quad (b)$$

From the equations ($a$) and ($b$), if we multiply the second by $a$, we get

$$\frac{ba^3}{3} + \frac{a^4}{4} + \frac{b^4}{12} = \frac{ba^3}{2} + \frac{a^4}{3} - \frac{b^3a}{6}.$$

Putting in that equation $b = H - a$, and solving it with regard to $a$, we obtain

$$a = \frac{H}{2}.$$

The centre of rotation is therefore, in this particular case, at half the height of the wall.

Let us now compute the case of a wall or column whose thickness is constant and, in view of its height $H$, negligible, while the direction of the impressed force, applied at its base, is normal to the length of the wall.

The force of inertia will have, for every element, the constant value

$$C_2 dx \cdot x\frac{d\omega}{dt},$$

and putting $C_3 = C_2\frac{d\omega}{dt}$, we have

$$Fa = C_3\left[\int_0^a x^2dx + \int_0^b x^2dx\right],$$

and integrating,

$$Fa = C_3\frac{a^3+b^3}{3} \qquad (c)$$

We have, besides,

$$F = C_3\left|\int_0^a x\,dx - \int_0^b x\,dx\right|,$$

*i.e.*

$$F = C_3\frac{a^2-b^2}{2} \qquad (d)$$

Equalising the equations (*c*) and (*d*), we get

$$\frac{a^3+b^3}{3} = a\left(\frac{a^2-b^2}{2}\right),$$

from which, $b$ being $= H - a$, results

$$a = \frac{2}{3}H,$$

which means that in the case of a wall of constant thickness the centre of rotation is at $\frac{2}{3}$ of its height.

In a like manner the centres of rotation of columns with other profiles can be determined.

Omori, putting together the data resulting from various earthquakes, found that, as regards long chimneys, the rupture takes place a little above the centre of rotation, which is at the same time a *steady point*, and he gives the following experimental formula:

$$a' = \frac{1}{2}\left\{a + \left(\frac{67}{100}\cdot H\right)\right\},$$

in which $a'$ represents the height at which the rupture takes place.

In order to determine the strength of our reinforced concrete wall subject to an impressed force, we shall apply—analogously to Omori's mode of proceeding with regard to chimneys[1]—the formula (8):

$$h=\frac{1}{2}\sqrt{\frac{M}{b}}$$

to the section of rupture of the wall as determined in the above manner. Once this section of rupture has been found, we can proceed as if we had to deal with a gradually applied force.

The calculation of high reinforced concrete walls, the thickness of which, as a rule, is very small in view of their height, can also generally be made in this manner.

*Numerical Example.*—Returning to the example already stated, let us suppose that the wall has a constant thickness $h=20$ cm.

The height of the section of rupture is given by the formula:

$$a'=\frac{1}{2}\left\{\frac{2}{3}\,7.10^2+0{\cdot}67.7.10^2\right\}=465,$$

r being $=\frac{2}{3}H$.

In our case the bending moment $M$ of the formula $h=\frac{1}{2}\sqrt{\frac{M}{b}}$ is given by the mass applied at the top of the wall and by the mass of the part of the wall above the section of fracture, to both of which the action of the earthquake has been gradually applied. The second of these forces is applied at the centre of gravity of the said part of the wall, *i.e.* at 117 cm. from the section of rupture.

The weight of that part of the wall is

$$235.100.20.25.10^{-1}\doteq 400 \text{ kgs.}$$

Consequently we have

$$M=\frac{400}{981}(1000.235+400.117)\doteq 115.10^3,$$

---

[1] F. Omori, *Publications, etc.*, p. 123.

and, substituting in formula (8) for the letters their values,

$$h = \frac{1}{2}\sqrt{\frac{115 . 10^3}{10^2}} = 17 .$$

The wall must, therefore, be 17 cm. thick.

If we call $x_2$ and $y_2$ the corresponding values of $x$ and $y$ for the section at the height $a_1$ of the parabolic wall, and applying formula (4), we get:

$$y_2 = 750 + 235 = 985 ,$$
$$x_2 = 10\cdot 2 ,$$

and that is to say that the wall at that height is 20·4 cm. thick.

It may without further demonstration be concluded that the parabolical wall of which we have given the calculation would resist even if the force of the earthquake should have to be considered as being applied impulsively.

Theoretically speaking, whenever the wall with which we have to deal is very high and of constant thickness, and not rigidly connected with a rather ample and deep foundation, it might also be overturned. But as a matter of fact very high structures, as we have seen, are never overturned; also, Omori does not even hint at high chimneys which had been overturned. In order to make that possible, it would in the first place be necessary that the amplitude of the movement of the soil assume much higher values than those which have been proved to occur in destructive earthquakes. Rather than being overturned, the chimneys will break. Thus, a circular chimney broke, the height of which was 30 metres, while its external diameter at the base was 4·30 metres. At any rate, by the use of suitable foundations every danger of the wall being overturned is excluded.

There is no need of other verification.

# CHAPTER VIII.

# FREE-WALL HOUSES.

Disposition of the Walls — Foundations — Orientation of the Walls — Height of the House — Bricks and Reinforced Concrete — Partition Walls — Roof and Floors — Applications.

The primary condition for walls of houses of this kind is that they can vibrate without impediment; it is advisable that they should be disposed as in fig. 36. The empty spaces at the angles have to be filled up with brickwork or also with reinforced concrete, which will adhere to the walls as long as they stand firm, but will be detached from them as soon as they begin to vibrate.

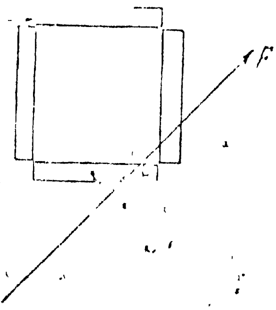

Fig. 36.

Seeing that these walls are intended to resist each one on its own account, it is, at least theoretically, not necessary that their foundations should invariably be bound together. In practice that will, however, be by far preferable, because the walls may not only be subject to stresses of flexure, or to stresses of flexure and shearing, which they are calculated to resist, but landslides may occur, or other phenomena which might alter the original distances or the reciprocal position of the bases of the walls elements which would probably cause the ruin of the building, notwithstanding the resistance of the walls themselves.

If there are no impediments to the free orientation of the building, and if the direction of the maximum intensity of the earthquakes in the place is known, it will be advisable to place the main walls at an angle of 45° to that direction, *F*. The reason is obvious. If we call *A* the maximum horizontal acceleration of the earthquake, the maximum acceleration by which the walls will be strained normally to their front side will be equal to *A* cos 45°, *i.e.* to about 0·7 *A*. This extreme case is more favourable than the other extreme in which the direction of the earthquake would be parallel to a wall. In the latter case there would be some walls strained normally by a maximum acceleration *A*, and some others would not be strained normally at all; and in such circumstances the collapse of some walls is evidently more likely than in the other case, the more so as the walls are independent of each other, so that it is even impossible to uphold the assertion (erroneous also in general, though advocated by some people) that, inasmuch as walls which are bound together mutually support each other, it would be advisable to make one part as secure as possible, as a support for the others which are more exposed.

As regards the height of the building, if the walls are parabolical ones, it must not, as a rule, exceed a certain limit given by the vibration period of the walls themselves. As has been seen, the latter have been calculated as not *infinitely* high columns whose oscillation period is very small compared with the period of the earthquake itself (or whose height is not *infinitely* great in ratio of its thickness or of the amplitude of the earthquake).

With brick walls it would in practice never be possible to arrive at a similar height, owing to the excessive dimensions which would have to be given to the base.

Very high walls would have to be calculated differently, that is to say, as if the force were suddenly applied.

We have refrained from developing this calculation for brick walls, because we do not consider it advisable to erect very high brick buildings in seismic regions.

As regards the use of reinforced concrete, it is, as we have seen, very suitable for this kind of work. In fact, the walls do not assume an exaggerated thickness, while having the required strength, and it is practically always possible to give them a suitable foundation to prevent their overturning. We have given the calculation of such walls also in the case that, owing to their dimensions, the seismic force would have to be considered as being applied suddenly. In quite analogous manner brick walls in the same conditions can, if required, be calculated.

The partition walls must be either so very light and weak that they do not disturb the vibrations of the main walls, or they must leave at the sides sufficient space to admit of all the walls vibrating. The empty spaces at the sides may be filled up in any convenient manner.

The roof must be free. That diminishes the bending moment of the walls and allows them to vibrate freely.

If there are floors, it is advisable that they should rest on beams which cross the thickness of the walls in such a way as to leave them free.

At the same time care must be taken that the beams protrude so much outside that they do not lose their support during the oscillations of the wall, even when they are of great magnitude.

Regarding some details of roofs and floors which are applicable also in this case, see Chapter X.

The form of the walls (very thick at the base when brick walls are in question), and the necessity of laying out the various parts of the building in such a manner that they can vibrate freely, are sure to prevent this mode of building houses being widely adopted. There are, however, cases in which it is the only mode which can be recommended, notably if the buildings to be constructed are of vast dimensions, such as large sheds, covered railway stations, theatres, etc., when, owing to their extent, the possibility or the convenience of a structure destined to behave, in face of an earthquake, like a monolith, is not to be thought of.

A small observatory with parabolic walls has been constructed in Japan,[1] and it has now for many years resisted well all shocks. It is built upon a basement of concrete, and has a superficial area of 83 sq. m. The walls are 5·50 m. high, and of thickness at the top of ·7 m. and at the base 2·40 m. The base of the walls is inserted into the basement and forms one and the same body with the concrete of the latter. The roof is free.

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[1] De Montessus, *loc. cit.*, p. 513; *Publications*, *etc.*, No. 20.

## CHAPTER IX.

# ON MONOLITHIC HOUSES IN GENERAL.

Rigidity of the Structure—Vibrations of the Walls—They behave like Reversed Pendula or Elastic Springs—Height of the House—Ground Plan—Junctions between the Walls—General Observations—Accessory Parts—Repairs.

THIS type of house seems to be more suitable for common use than free-wall houses. The difficulty lies, however, in enabling them to vibrate like a single piece. It is therefore necessary that their component parts should be bound together as much as possible in a single homogeneous entity, and subjected, as far as it can be done, to the same stresses. Such houses ought not to be of very large dimensions, but of compact form and based upon a very solid continuous foundation. Omori,[1] during various earthquakes, took the opportunity of making observations and examinations regarding the manner in which the walls of the building of the Engineers' College at Tokyo (a two-storied building of which the external walls are about 10 m. high) were affected. He found that when the oscillations were slow, *i.e.* of about 0·5 second, they were the same on the upper and on the ground floors; but when they were rapid, their amplitude under the roof was double of what it was on the ground floor.

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[1] F. Omori, *Publications, etc.*, No. 4, p. 10; *Note on Applied Seismology, etc.*, p. 342.

Seeing that the period of vibration was the same in both cases, it seems that in violent shocks the wall upon which the roof rested behaved like a reversed pendulum subjected to forced vibrations, its motion being synchronous with that of the earthquake. A similar wall has been defined by Omori as seismically *ordinary*. On the other hand, a wall is *worst* which oscillates at the top several times more amply than at the bottom, like an elastic spring vibrating within its own period whatever may be the period and the amplitude of the movement of the soil (as a wall of the Museum of Natural History at Tokyo). *Good* is a wall which has the same movement as that of the soil, at the top as well as the base (as a wall of a bank building in Tokyo).[1] Omori also observed that during destructive earthquakes the damage to two-storied buildings is generally restricted to the upper floor, a fact which is certainly to be ascribed to the increase in the vibrations in the highest part of the building.

It is therefore advisable to construct very low buildings with very few stories and a light roof which cannot be deformed and is rigidly united with the walls. In addition to that, the various parts ought to show as few interruptions of continuity as possible, and the transition from one to the other ought never to be made in a sudden manner. The circular form is theoretically the most suitable one for a building, the more so as, when using it, no account need be taken of the direction of the seismic action, which is very often ill-defined or wholly unknown. The good resistance of circular buildings to earthquakes has, besides, already been observed in practice.[2]

Large curves of junction generally ought to bind together all walls of different directions. Figs. 37 and 38 represent two rough ground-plans of houses built according to that principle. In them the dotted lines may represent walls of reinforcement across a passage. Polygonal plans, short rectangular plans, etc., may also be adopted according to the practical needs.

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[1] *Publications, etc.*, No. 20.

[2] De Montessus, *loc. cit.*, p. 496.

The rooms ought generally to be small; windows and doors, too, must be small and few in number; the walls well bound in, the floors light and well bound to the walls (vaultings must be altogether avoided), and so on. Regarding certain details of construction we shall give a few hints later on.

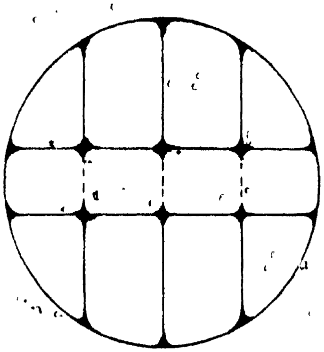

FIG. 37.

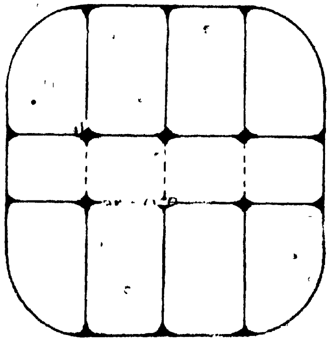

FIG. 38.

As regards the orientation of the house, it will always be well, even for internal partition walls, to pay attention to what we have said in Chapter VIII. In these buildings it is also necessary to avoid the construction of parts which are not true members of the real body of the building itself. Balconies, ornamentation of the walls, crownings or copings, high chimneys, towers of any kind, entrance porches, caryatides, and anything else of that sort ought to be suppressed.

## ON MONOLITHIC HOUSES IN GENERAL.

The repairing of old houses for the purpose of giving them a form which is better adapted to resist seismic shocks is generally not advisable, owing to the difficulty of attaching the new things to the old ones in such a manner that all taken together form a homogeneous entity. Experience proves that during an earthquake the old and the new are torn asunder.

It will be understood that a type of house like those of which we are speaking, even though they be constructed of bricks (provided that workmanship as well as materials are of the very best), presents a good guarantee of solidity, the more so when it is borne in mind that, as we have already stated, even ordinary building structures, if of good materials and workmanship, will generally offer a remarkable resistance to earthquakes.

## CHAPTER X.

# GENERAL TEST CALCULATION OF A MONOLITHIC BRICK BUILDING.

Vibration Periods—Statically applied Force—Test Calculation of one Type of Houses—Conditions for Monolithism — Basement — Maximum Vertical Acceleration—Specimens of Basements of Reinforced Concrete—External Walls—Internal Walls—Floors — Tying and Anchoring — Windows and Doors — Staircases — Roofs—Chimneys.

LET us assume that the type of house we wish to calculate has a ground floor, a first floor, and attics, its elevation being represented roughly by fig. 39, and its ground plan by fig. 37. For simplicity's sake, we shall include in our computation only the external circular wall and the weight of the floors and roof. First, we will have to find out whether the maximum acceleration will have to be understood as being applied gradually or impulsively.

Omori[1] made with his oscillating table experiments regarding the periods of vibration of cylindrical and prismatic columns whose height varied between 1 and 2 metres, while their thickness was from about 10 to 20 cm. He found, as already mentioned, that the vibration period stood approximately in inverse ratio to the thickness, and in direct ratio to the square of the height, of the column, both for massive and for hollow columns. For the latter ones the external thickness is measured.

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[1] F. Omori, *Publications, etc.*, No. 4, p. 119.

From experiments made in Japan during an earthquake with a chimney, it furthermore results that the vibration period of the chimney increased somewhat with the increasing amplitude of the vibration. Thus, with a chimney 5·90 m. high, of which the square section had sides of 0·81 m., the mean period of vibration was in

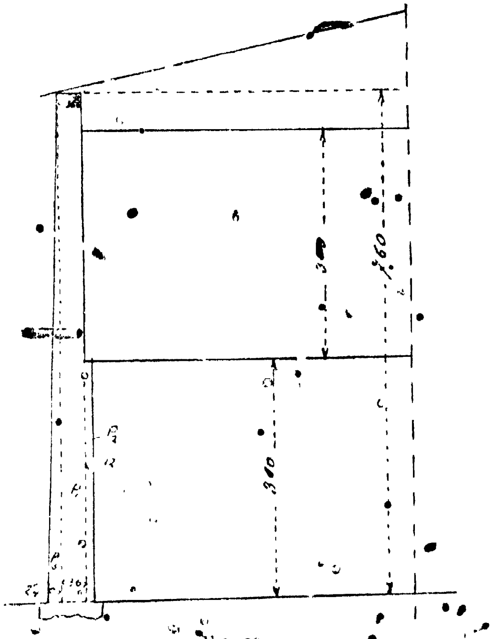

FIG. 39.

one experiment 0·55 second, and the period of the maximum movement was 0·85 second (the magnitude of the maximum movement at the top of the chimney was 90 mm.). In another case the mean period was 0·62 second, while the period of the maximum movement was 0·99 second (the magnitude of the maximum movement at the top of the chimney was 1·62 mm.).

The conclusion to be drawn from these experiments is, that the vibration period of the chimney in question during a great earthquake would be about 1 second.

The height of the house is not much greater than that of the chimney, while its thickness by far exceeds that of the latter. If it were admissible to apply to this house the results of Omori's experiments with columns, it would follow that its proper period would be about one-tenth of a second. At any rate, there can in this case be only a question of a very short vibration period, and we may assume without further consideration that in the case of a destructive earthquake whose period is from 1 to 2 seconds the force may be understood as being applied gradually.

Let us now determine this force $F$ with reference to fig. 39. We assume the permanent and accidental loads on the floor of the first story to be 330 kgs. per sq. m., those on the roof and the second floor 300 kgs., and we understand the force to be applied at 7·60 m. from the ground. The weight of the brickwork is 1600 kgs. per cub. m.

Let us take $P_1$, $P_2$, $P_3$, $P_4$, $P_5$ as the weights of the three parts into which we have, according to the figure, divided the external wall, *i.e.* its section, and the total loads of the floors and the roof. We then have:

$$P_1 = 0\cdot36 \,.\, 7\cdot60 \,.\, 2\pi \,.\, 5\cdot30 \,.\, 1600 = 145,000 \text{ kgs.}$$
$$P_2 = 0\cdot12 \,.\, 3\cdot60 \,.\, 2\pi \,.\, 5\cdot06 \,.\, 1600 = 22,000 \text{ ,,}$$
$$P_3 = 0\cdot5 \,.\, 0\cdot24 \,.\, 7\cdot60 \,.\, 2\pi \,.\, 5\cdot60 \,.\, 1600 = 51,200 \text{ ,,}$$
$$P_4 = \text{about } 100 \,.\, 300 = 30,000 \text{ ,,}$$
$$P_5 = \pi \,.\, 5\cdot12^2 \,.\, 330 = 27,000 \text{ ,,}$$

The greatest bending moment $M$ is acting upon the base, and the section of rupture will be the base of the solid. Let $P$ be the total weight of the house, and $h$, $h_1$, $h_2$, . . . $h_5$, the distances from the base of the points in which $P$, $P_1$, $P_2$, . . . $P_5$ will be applied. We then have:

$$P = P_1 + P_2 + P_3 + P_4 + P_5 = 275{,}000 \text{ kgs.}$$
$$Ph = P_1h_1 + P_2h_2 + P_3h_3 + P_4h_4 + P_5h_5.$$
$$275 \cdot 10^3 h = 10^3(145 \cdot 380 + 22 \cdot 505 + 30 \cdot 760 + 27 \cdot 360).$$
$$h = 425 \text{ cm.}$$

Supposing, therefore, that the house will behave like a single elastic solid, we call $A$ the area of the section of the base, $z$ the distance of the farthest fibre from the neutral axis, $I$ the moment of inertia, $\sigma_1$, $\sigma_2$ the stresses in the two fibres of the section which are farthest distant from the neutral axis. We then have:

$$A = \pi(572^2 - 500^2) = 210 \cdot 10^3 \text{ sq. cm.}$$
$$M = Ph\frac{a}{g} = 275 \cdot 10^3 \cdot 425 \cdot \frac{400}{981} = 48 \cdot 10^6.$$

The modulus of resistance for one hollow circular section with external diameter $D$ and internal $d$ is, as is known, expressed by

$$\frac{I}{z} = 0{\cdot}098\frac{D^4 - d^4}{d}.$$

In our case

$$\frac{I}{z} = 0{\cdot}098\frac{1144^4 - 1000^4}{1144} = 61 \cdot 10^6.$$

Substituting in formula (7), we have:

$$\sigma_1 = 2{\cdot}9 \text{ kgs./sq. cm.}, \qquad \sigma_2 = 0{\cdot}51 \text{ kgs./sq. cm.}$$

$\sigma_1$ and $\sigma_2$ are both compressive stresses. We have, therefore, a strength which is more than sufficient.

It ought to be observed that, by not making allowance for the effect of the internal walls of the house, it is not impossible that we are doing something to the prejudice of the stability. That would certainly be so if we had not in our calculation taken account of the weight of the roof and floors, and had only considered the great hollow column formed by the external wall, as may be easily derived from the formula (1), Chapter VI., and which is, besides, obvious. In fact, if we have a cylindrical or prismatic column of a given height, the bending moment acting upon the base is, in our case, proportionate to the mass; while, on the other hand, the modulus of resistance grows the more the mass gets distant from the centre and nearer to the periphery, which means that in the case of the house the internal walls represent matter less well utilised for the resistance to flexure. In view of the margin which is still left to us, before $\sigma_1$ and $\sigma_2$ reach the value of the strength of the material used, we do not require further calculations.

Nor will there be, at least theoretically, any danger of sliding, because, as we have seen already in Chapter VI., the frictional resistance of the brickwork is superior to the horizontal force applied in every joint. This danger appears also smaller, if it is borne in mind that, before an earthquake occurs, the mortar will have a certain time in which to get hard, and, consequently, the mortar too will enter into action (in contradistinction to what might happen to walls destined to support embankments of earth).

A building like that which we have been considering would, therefore, be theoretically well fitted to resist earthquakes. Let us now see, by going a little more into details and examining the principal parts, how we can manage to make the building behave like a monolith.

*Basement.* — The basement must be so laid down that it remains level; keeps the bases of the walls well together; conveniently divides the weight of the building over the ground; and distributes uniformly the stresses which, during an earthquake, the ground, transmits to the various parts of the building, so that all of them are, as far as possible, strained in the same manner. When it is constructed so that it will resist well considerable vertical forces, it will also resist well the horizontal acceleration.

As we have seen in Chapter I., the vertical component of the movement during the earthquake of Tokyo on the 20th June 1894 was 50 mm. If we in our calculation make allowance for a component of 100 mm., we certainly anticipate a very strong earthquake.

We do not think it necessary to take account of the visible waves of gravity, seeing that building on ground where they can be observed must be avoided.

Assuming that the period of a very intense earthquake is 2 seconds, the maximum vertical acceleration $A$ would, if the extension of the vertical movement be 100 mm., be given by

$$A = \frac{4\pi^2}{T^2}a = \frac{4 \cdot 3{\cdot}14^2 \cdot 50}{4} = 500 \text{ mm.}^1$$

The maximum vertical acceleration would, therefore, be 500 mm., and that is not very important in face of the acceleration due to gravity.[2] We put it down, without other consideration, that the reaction of the soil is equal to $1{\cdot}5P$, $P$ being the weight of the building. In so doing we proceed as is usual in constructing ordinary buildings when they are subjected to a dynamic load.

When the ground is very solid and compact, and when no surprises are to be feared in cases of earthquakes, one might even,

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[1] It appears that, though the horizontal and the vertical movements of an earthquake generally have the same period, it is not necessarily the case (Dairoku Kikuchi, *Publications, etc.*, No. 19).

[2] During the earthquake of 1891, it seems that the maximum vertical acceleration was, as we have seen, at Nagoya 800 mm./sec.$^2$, and at Gifu about 950 mm./sec.$^2$

relying upon the margin left by the safety coefficient which is generally adopted in building, calculate the reaction of the soil equal to $P$. It is, however, advisable to be very cautious in so reducing the strength of the basement, because the latter is the best guarantee for the solidity of the building.

As material for the construction of the basement it will be best to stick to reinforced concrete, and inasmuch as, for its calculation, it would be necessary to enter into details of construction which are not within the scope of this work, our readers are referred to special treatises on the subject.[1]

We shall only say in general that the basement must be calculated and constructed like a reversed floor, *i.e.* the slab itself must be laid out flat on the ground, with the secondary and principal ribs on top. It must, in fact, resist the reaction of the ground directed from the bottom to the top.

From some instances which will be mentioned later it results that the thickness of a basement will in actual cases never assume exaggerated values, and that, therefore, the use of similar foundations is most advisable.

On a general basement were founded[2] the corn magazines of the harbour of Genoa, which are constructed of reinforced concrete. That basement consists of a sort of reversed floor, and is composed of a general foundation slab 25 cm. thick, with secondary ribs of 65 by 25 cm., distant from each other (axis to axis) 2·666 m., and with principal ribs of 75 by 30 cm., distant from each other 3 m. (also axis to axis). In this manner the pressure on the soil is reduced to 1·6 kg. per sq. cm.

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[1] See, for instance, C. Guidi, *op. cit.*

[2] C. Guidi, *loc. cit.*, p. 31.

In the basement of the Sampiardarena mill the thickness of the foundation slab varies from 20 to 30 cm.; the ribs on top of it protrude to a height which reaches 90 cm. in correspondence with the bases of the pillars. In the building of the corn magazines the pillars stand at distances of 2·50 m. from each other in one direction, and of 3·50 m. in the other, and the maximum load transmitted by the basement to the ground reaches 25,000 kgs. per sq. m.

Also where there is no danger of earthquakes, but where the ground on which a building has to be erected is friable while underneath it at considerable depth there is compact soil, it would be advisable, from the point of view of safety, to place the foundation upon the compact soil and that can be done very well by lines of piles of reinforced concrete, upon the heads of which the basement would be placed.

This kind of foundation has been very widely adopted during the last few years. Thus, at the principal railway station at Hamburg[1] 580 piles were rammed into the ground. These piles were from 5 to 12 m. long, and 36 cm. wide at the sides, reinforced at the angles by four irons 25 mm. in diameter, and bound by 8-mm. wire at distances of 25 cm. Every pile that was sunk was able to carry a load of 50 tons.

At the new railway station at Metz 3000 piles of reinforced concrete, and with hexagonal or pentagonal sections, were rammed in. The piles were from 20 to 16 m. long, with sections of from 1200 to 1600 sq. cm., and they could support in all safety from 58 to 65 tons each.

The construction on lines of piles sunk through loose ground until the solid ground has been reached will obviously be still more advisable, if not absolutely necessary, if one is compelled to build in seismic regions on similarly dangerous ground.

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[1] C. Guidi, *loc. cit.*, p. 33.

*Walls.*—It is advisable that the external walls should slope at the outside. So, as they are generally built at present, with steps towards the interior at each floor, the line of direction of the wall does not pass through the centre line of the base, but is rather pushed to the outside. This is not good for stability, because the wall can more easily be bent or overturned at the side which is free. Even if it is not considered convenient altogether to abolish the steps to the inside, the disadvantage connected with them can always be corrected by the outside slope.

At the angles the walls ought to be united between each other by large curves of union.

The internal walls must not be merely united with the external walls by being placed against them, but they must be so constructed that they all form one piece with them.

The walls ought not to be weakened by chimney flues, etc. The latter must be arranged so that they correspond with the curved parts of the walls, which assume the form of large pilasters (figs. 37, 38). These pilasters may very well be left hollow.

*Floors.*—The vertical component of the seismic movement will have the effect of making the floors vibrate. Much account, however, need not be taken of that in the calculation, partly in virtue of their safety coefficient and partly owing to the small importance generally possessed by the vertical component of the movement.

It is advisable that the floors should be very light. The beams (of iron) must be very well imbedded in the walls, and some of them must act as ties by protruding outside the walls. The bolts must in that case be very long. In order to have ties at angles of 90° between each other, the beams of one floor ought to be placed at angles of 90° to those of the other.

Vaulting must be absolutely avoided.

*Tying and Anchoring.*—The solidity of the house ought as a rule to be guaranteed by a system of solid iron ties placed, for instance, in the following manner:—At the height of each floor and of the roof there might run in every wall a tie, and the heads of the ties in every wall might on both extremities of the wall be united together by means of a vertical bar which would act as a common bolt.

If the external walls end in large curves of union, it is advisable to run ties also along the latter.

*Windows and Doors* must be as few and as small as possible, in order not to weaken the walls, and each one ought to be surmounted by a full-centre arch of vaulting. The doors ought, for the sake of safety, to open to the outside.

*Staircases* ought to be, wherever possible, winding staircases, and in any case they ought to have the steps well imbedded in the wall on both sides.

*Roof.*—The framework must form an indeformable whole. If a roof-truss of any kind has to be constructed, its single parts must be rigidly joined together. The woodwork must be bound and tied as shown in figs. 22 and 27. The roof must not weigh heavily on some points of the walls, but its weight must be distributed over their whole length. That can be done by making a frame of beams run on the crest of the walls, and fixing the roof on to them. In this manner the rigidity of the roof will also be increased.

The roof covering must be as light as possible and fixed to *the frame. Among tiles, those called* "Marseilles tiles" *must be recommended.*

Generally, however, those houses for which we have recommended small doors and windows and no balconies, and the inconsiderable height of which admits of the upper parts being reached by mounting only a few steps, might very conveniently be covered by a large terrace-roof, constructed, of course, of light, though strong, materials.

*Chimneys* ought not to be raised above the level of the roof by more than a few courses of bricks. If it is necessary that they should be higher, the part protruding above the roof ought to be made of galvanised iron plate. Chimneys do not, as a rule, resist earthquake shocks; even if the latter are weak, and they break at the base.

# CHAPTER XI.

# SOME NOTES ON THE CONSTRUCTION OF MASONRY.

Use of Bricks—Use of Stones—Quality of the Bricks—Japanese Bricks—Mortar—Fresh and Salt Water—River Sand and Sea Sand—Lime—Mixture of the Ingredients—Cement—Season for Building—Precautions—Interruption and Resumption of Working.

WITHOUT in the least wishing to lay down real standard rules, a few hints regarding those points upon which the attention of the builder ought to be chiefly directed will not be out of place, remarks being limited to the use of bricks as principal material. The use of stones is not, in the author's opinion, advisable, unless they are blocks specially squared in order to admit of being fitted into one another for better resistance to shearing stresses. At any rate, their use always entails the inconvenience of a greater weight.

As a matter of principle it must, further, be plainly understood that to increase the thickness of a wall is by no means a remedy against the break-down of the material, because by an increase of the thickness the mass is increased, and consequently also the force applied by the earthquake.

The bricks must be burnt first to the right point, because, if over-burnt, the mortar does not adhere to them, and in building structures of this kind the adherence of the materials is of the greatest importance. They must have rough faces, and when they are made with the wire moulding machine (and not in sanded moulds) they must be purposely ribbed, for otherwise their faces would not be sufficiently granulous.

For the upper parts of buildings, the use of hollow bricks seems advisable on account of their lightness. Care must be taken, of course, to avoid the mistake that, owing to an excessive discontinuity in the structure of the walls, the house should ultimately consist of two monoliths, one placed upon the other, instead of forming a single entity.

The Japanese advise the employment of bricks of such a form that they can be imbedded into each other in order to render impossible any slipping out. The bricks are keyed to one another as in fig. 40, according to Milne.[1]

The mortar must be prepared with clear and chemically pure water. With lime the use of sea water must be avoided. On the other hand, there are cements which give good mortar with salt water.

The best sands are those of siliceous nature; they ought to be coarse-grained, the single grains to be large and sharp-cornered, and not of rounded contour. Furthermore and above all, they must be pure, *i.e.* they must not contain earth or be mixed with organic substances. River sands are very good. Sea sands ought, before use, to be freed from their salt contents—left a long time exposed to the air and well washed.

Good lime ought to be used, the better of hydraulic; and the mortar ought to be prepared from time to time, so as to be always fresh when used, *i.e.* before it begins to harden. Mortar ought not to be hard, especially in summer time; the bricks absorb so much the more water, to the advantage of a firm setting. As regards the mixture of the ingredients, there should be about 300 or 350 kgs. hydraulic lime to the cubic metre sand for open-air work, and up to 400 kgs. for masonry under water. A mixture of ordinary lime and puzzuolana may also be used with advantage in preparing mortar. This was done by the ancient Romans with the best results, as is universally known.

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[1] De Montessus, *loc. cit.*, p. 479.

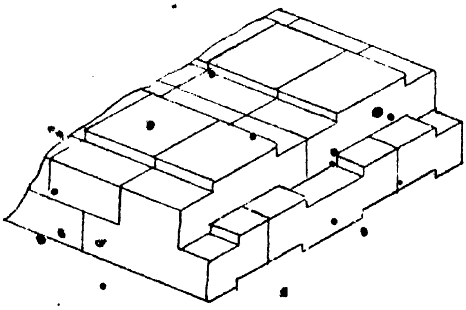

Fig. 40.

Cement mortar gives the greatest rigidity and strength to masonry. Mixtures of cement, lime, and sand also give excellent results. But the cost of cement and the great difficulty experienced in its use make the latter not so extended by far as the use of lime, at least in Italy.

It is advisable not to build when the weather is either too hot or too cold, as the mortar will not adhere very well. In summer time the walls in course of construction ought to be, if possible, protected by matting, or water ought to be poured over them from time to time, so that they do not dry too quickly. In winter time the upper parts should be covered as a protection against the inclemency of the weather.

Sometimes it happens that a building is not constructed at a stretch, but with one or more interruptions and resumptions of the work. In that case it is above all necessary to manage in such a way that the new is perfectly well attached to the old; otherwise the seismic solidity of the building would be gravely jeopardised. On resuming the work in such a case, the old plaster and mortar ought to be thoroughly scraped off the surface of that part of the already erected portion to which the new additions have to be attached, and, after thoroughly repairing all defective points on it, the surface should be covered with a coat of liquid lime or cement.

## CHAPTER XII.

# ON THE STABILITY OF AN ORDINARY HOUSE DURING AN EARTHQUAKE.

How the House is affected—Every Part vibrates on its own Account—Examples—Cracks and Ruin—Houses ruined by the last Earthquake in Calabria—Rupture Line of the Walls—Stability of a House, of the Junctions between its Parts, of an External Wall—Stability with regard to the Height of the House—A Building at Nagoya—Stability and Maximum Acceleration—Criterion of the Stability of a House.

THE result of the calculation made in Chapter X. is important, because it proves that a house which behaves like a monolith cannot, generally speaking, be destroyed by an earthquake. Let us now see how an ordinary house resists the action of the seismic forces.

Before all, it is certain that such a house in an earthquake does not behave like a monolith, because, if it did, it would not be damaged or ruined. The house will, on the contrary, tend to divide itself into several parts, each of which will vibrate on its own account. There will certainly be shocks between the various parts vibrating in discord, especially in the upper stories, where the vibrations are of greater amplitude, and the house will disintegrate more and more; the disintegration will take place along lines of weakness and according to the direction of the seismic force. Generally each one of the walls will vibrate on its own account, and floors as well as roof will do the same.

Taramelli and Mercalli[1] mention the case of a house at Oneglia of which three ceilings fell down because the walls, by vibrating, failed to support them any longer. The house was thus converted into a sort of well formed by its external walls. Numerous analogous cases were observed during the earthquake in Calabria and Sicily in 1908.[2] Omori observed, as we have seen, that the walls of the Engineers' College at Tokyo acted like reversed pendula, and those of the Museum of Natural History like springs fastened at one end. The great majority of common walls will vibrate in one of these two modes. The resistance of the walls is, therefore, of greater account to the house than that of the floors and the roof, because it is the former which support all. The effect produced by the floors and the roof upon the walls is that, by not vibrating synchronously with them, they tend to ruin them. Omori attaches, in fact, great importance to the action of the roof in the destruction of brick buildings.

Under the action of an earthquake the various parts of a house thus begin to vibrate, and by repeated shocks in various directions, by being jolted against one another, by possible subsidences of the ground, etc., they will be separated from each other, and the house will crack. If the seismic action continues in sufficient strength, all or some parts of the house will be destroyed.

We reproduce here some photographs from the disastrous earthquake in Calabria on the 28th December 1908. According to our opinion, these photographs, regarded jointly, can give a sufficiently clear idea of one of the modes in which the destruction of houses seems to occur.

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[1] De Montessus, *Beiträge zur Geophysik*, vii. p. 195.

[2] Omori, *Bulletin*, etc., vol. iii., No. 2.

[*To face p.* 100.

FIG. [illegible]1.

## ON THE STABILITY OF AN ORDINARY HOUSE.

In fig. I. of the appended plate it is seen that by the seismic shocks and by the jolts received from other parts of the building an external wall has been detached. If the action of the earthquake had continued with sufficient intensity, the wall would have been destroyed.

Fig. II. of the appended plate shows an external wall which has been destroyed.

From fig. III. it results that it is not on account of the roof that the house is destroyed—a fact also shown by a very great number of photographs which we cannot reproduce here.

It is noticeable that very often the walls of a house are destroyed, either at half their height, or else completely or almost completely, while another part of the same house may remain standing undamaged. It seems impossible generally to attribute to the roof a similar destructive effect, even if the roof is wholly or partially destroyed. During an earthquake the various parts of a house have, therefore, a tendency to disintegrate and then to be destroyed.

Inasmuch, however, as before being destroyed, they are generally disintegrated from each other, each one will go to destruction vibrating as if it stood alone. That may explain how the rupture of the external wall in figs. I., II., III. occurred at the height of the first-floor windows.

In fact, as we have seen, we have here columns whose vibration period is sufficiently long, and the seismic force must be understood to have been applied suddenly. The point of maximum stress and, therefore, of rupture is, in that case, theoretically proved for walls of constant thickness to be at two-thirds of their height, and for walls of triangular profile to be at one-half of it. Ordinary walls of a house have a certain thickness also at the top, but they are thicker at the bottom; they are therefore intermediates between these two types. The section of the wall at the height of the window sills on the first floor is the weakest in the vicinity of the theoretical section of rupture. The wall must, therefore, practically break there.

Sometimes it happens that certain high walls are destroyed down to the bottom. That can be explained as follows:—The wall breaks first at about two-thirds of its height, and once it is broken, the part that has remained standing assumes a vibration period of its own which is so short that the seismic force may be considered to be applied statically at its centre of gravity. If, therefore, the earthquake continues, the second rupture must take place at the base of the wall.

It now remains to gain an idea of the value of the stability of a house which does not behave like a monolith, *i.e.* of an ordinary house.

That stability generally depends mainly on the manner in which the walls are joined together, on the lack of synchronism in the vibrations of the various members of the house, and on the stability of the walls themselves (cases of displacements of the ground or of the foundations being excluded).

The stability of the walls is easily calculated. Not so the resistance of the junctions of the walls, which act favourably to the stability of the house, or the effect of the lack of synchronism in vibration, which act in the opposite direction.

We only observe that the maximum amplitude of the vibrations at the top of a column increases with the increasing height of it, and that, the more ample the vibrations are, the less resistant will be the junctions in any point of a building, and the more powerful the effects of the lack of synchronism in the vibrations of its various parts.

On the other hand, with the growing height of a building increases also the importance of the junctions of a wall to the rest of the building itself. The total effect of the lack of synchronism between the vibration of the wall and that of the other parts of the house in contact with it also increases.

In these circumstances, and after what we have said about the mode in which a house is destroyed, the supposition is naturally that in an ordinary house the effect of the resistance of the junctions and that of the lack of synchronism in vibration might, at least within certain limits, neutralise each other, and that the stability of the structure of the house might be considered equal to that of the main wall itself which is first destroyed, *i.e.* that of an external wall, the external walls being those which, the resistance itself being equal, are most easily destroyed.

Let us now consider a case mentioned by Omori,[1] viz. the post and telegraph office at Nagoya, destroyed by the earthquake on the 28th October 1891. That was a two-storied brick building, and—as far as can be judged from a photograph—the examination of certain details shows that the height of its external walls was 9 metres. The rupture of the walls occurred, apparently, at 5·50 m., the height of the first-floor window sills. The thickness of the wall, considering the dimensions of Japanese bricks, was probably 45 cm. From the photograph it further appears that the empty spaces of the windows were in length about one-third that of the wall.

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[1] F. Omori, *Publications etc.*, No. 4, p. 11.

The direction of the earthquake was normal to the ruined walls, and, from measurements taken, resulted $A = 2600$ mm. per sec. per sec.

If we call $a$ the stability of the above wall, we get from formula (2), substituting and taking $R = 3$ kgs./sq. cm.,

$$a = \frac{45 \cdot 981 \cdot 3}{2.16.10^{-4}.(12.175)^2} = 340.$$

Making allowance, regarding the resistance of the section of the wall, for the empty spaces of the windows (we ought really also to make allowance in the calculation for the bending moment, but that's of less importance), we find that the moment of inertia for the section itself must be reduced by $\frac{1}{3}$, and, therefore, the value found for $a$ by the same quantity. The result is therefore:

$$a = 340 . \frac{2}{3} = 227,$$

which means $a = 2270$ mm. per sec. per sec.

This value of $a$ would be very much like the true one also as regards the stability of the building, but it is merely hypothetical. At any rate the true value of the stability of the house must be of the same order, and we shall conclude by saying that the resistance of a brick house may, by way of gross approximation (except the case of simple destruction of the roof with its consequences, and except also generally those cases of partial destruction in which the framework of the house remains standing intact), be reduced to that of one of its external walls separately considered.

This—taken for what it is worth—would be a mode of obtaining an idea of the stability of an ordinary house in an earthquake.

## CHAPTER XIII.

# GENERAL TEST CALCULATION OF A MONOLITHIC HOUSE OF REINFORCED CONCRETE.

Particular Case—Graphic Calculation—Maximum Stresses of the Concrete and the Iron—General Hints regarding Construction—Typical Precautions—Filling up with Masonry.

LET us state a specific case, assuming that the house has a square base and that its vertical section is represented by fig. 41. The

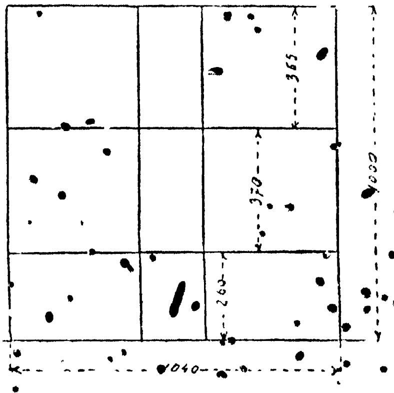

FIG. 41.

width of the external walls is 18 cm., and they are reinforced by iron rods placed vertically (bars of resistance), and by round iron rods placed horizontally (bars of distribution), which latter we do not consider here. The floors as well as the roof have a slab of 10 cm. thickness and ribs; their overburden is 300 kgs. per sq. m. The weight of reinforced concrete being 2500 kgs. per cub. m., the actual weight of 1 sq. m. of flooring or roofing, including the ribs, will be about 300 kgs., and thus the total weight of floors and overburden will be 600 kgs. per sq. m.

If we call $P_1$ the total weight of the external walls, and assume the length of one external wall to be 10·40 m., measured at the outside, we have:

$$P_1 = 4 . 10 . 22 . 0{\cdot}18 . 10 . 2500 = 185{,}000 \text{ kgs.}$$

If $P_2$, $P_3$, $P_4$ are the weights of the floors and the roof, we have:

$$P_2 = P_3 = P_4 = \text{about } 100 . 600 = 60{,}000 \text{ kgs.}$$

For the sake of simplicity we shall in calculation omit the internal walls (which really are of service in stability, not only directly as supports of the floors, but also indirectly as ties between the various parts of the house).

If we call $P$ the total weight of the house, $h$ the height of its centre of gravity, $h_1$, $h_2$, $h_3$, $h_4$ the heights of the points in which $P_1$, $P_2$, $P_3$, $P_4$ are applied, and taking into account the dimensions of the house as they result from the figure, we get:

$$P = P_1 + P_2 + P_3 + P_4 = 365{,}000 \text{ kgs.}$$
$$Ph = 365 . 10^3 . h = 10^3 [60(260 + 630 + 995) + 185 . 500] = 10^3 . 205{,}500 \text{ kg./cm.}$$
$$h = 560 \text{ cm.}$$

Analogously to what we have said in Chapter X., suppose that to the centre of gravity of this house, which behaves like an elastic body of one single piece, there be applied gradually a horizontal force equal to $P\frac{a}{g}$, $a$ being = 4900 mm. per sec. per sec., and directed from right to left, parallel to one side of the house. The force resulting from the various forces acting upon the house strikes its base in a point $X$ distant by $h\frac{a}{g} = 225$ from the centre of magnitude of the base.

In fig. 42 the section of the base of the external walls of half the house is represented, the scale of reduction being 1 to 128.

The section may be considered to be strained by a vertical eccentric stress $P$ applied in $X$. This stress is, indeed, equivalent to a vertical force $P$ applied in $G$, the centre of gravity of the base, and to a couple of moments $Pd$, $d$ being $= GX$. But $Pd$ is $= P\frac{a}{g}h$, and consequently it is apparent that the section of the base of the house is strained by the weight $P$ of the house itself, and by the horizontal force of the earthquake $P\frac{a}{g}$ applied at the height $h$ from the base.

Divide the section of the walls into so many "bands" of the same size which are normal to the axis of stress and to their centres of gravity. Apply[1] forces proportionate to the areas reduced at the base $a = 50$ cm. The sections of the rods must be imagined to be concentrated in points distant by 1 metre from each other, and the sections so obtained have to be multiplied by the ratio between the modulus of elasticity of the iron and that of the concrete (ratio which we may put down $= 10$), then they have to be reduced to the base $a$, and to their centres of gravity forces proportionate to them have to be applied.

At the right hand in the figure the line of the forces is marked. From 0 to 12 are marked the forces relating to the sections of the

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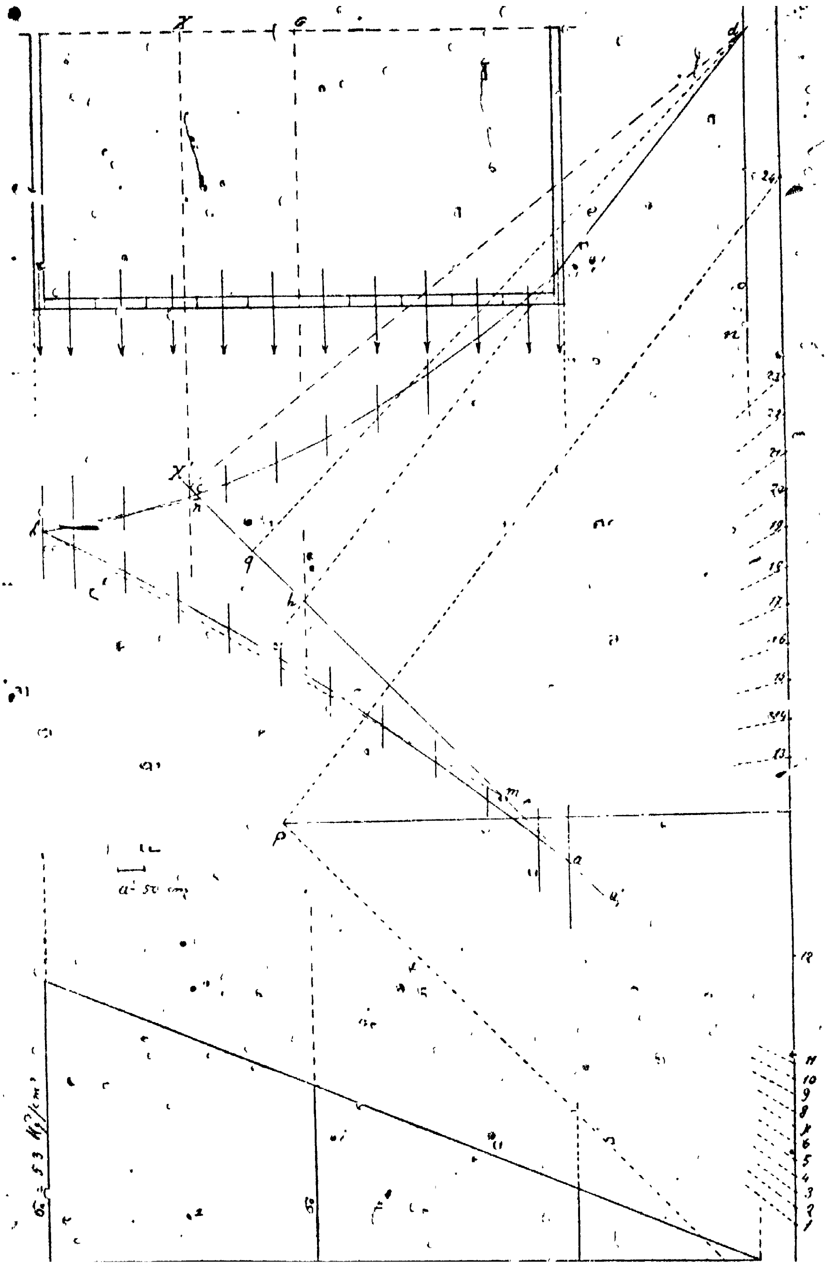
[1] C. Guidi, *loc. cit.*, p. 88.


X
G
d
e
f
X'
n
k
q
h
c'
m
p
a
a = 50 cm
σ₀ = 53 Kg/cm²

irons, starting from the right and going to the left. From 13 to 24 are marked the forces corresponding with the bands of concrete from left to right.

Let us suppose that for every metre run of wall there are 7·5 sq. cm. of section of vertical rods, and that the four corners are reinforced each by 12·5 sq. cm. of iron section. The walls normal to the direction of the shock and placed vertically in the figure, represent each one band, in each of them the section of the concrete is 18,700 sq. cm., and that of the iron reduced to concrete 10,000 sq. cm.; the forces representing iron and concrete have the same point of application.

For every horizontal band of wall we have 1800 and 750 sq. cm. respectively—figures which we have doubled in the diagram because the bands of the two horizontal walls correspond with each other, and thus the figures resulting from the diagram stand each for a couple of bands.

There is, further, constructed relatively to the pole $P$ the funicular polygon of these forces.

On its first side $aa'$ is to be marked X', in correspondence with X. It is known[1] that, by drawing from the point $X'$ the straight line $X'd$ so that the latter is the line of equalisation regarding the funicular polygon $abcd$, the point $d$ determines the position of the neutral axis $n$. The funicular polygon $abc$ is to be substituted by the broken line of equalisation $amn'n$. The area of that polygon will, therefore, be equal to that of the triangle $mbn$. Then $d$ is determined so that—$dq$ being the line of equalisation regarding the polygon $dc$—the area $X'dq$ is equal to the area $mbn$.

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[1] C. Guidi, *Lezioni sulla Scienza delle Costruzioni*, Part II., 4th ed., p. 124.

In our case the neutral axis lies outside the figure. That means that the reinforced concrete which forms the walls of the house is only subjected to compression. If we call $\sigma_0$ the mean unitary pressure to which the concrete is subjected, and $A$ the ideal area, *i.e.* converted into concrete, of the horizontal section of the external walls of the house (a compressed section), we have

$$\sigma_0 = \frac{P}{A} = \frac{365.10^3}{108.400} = 3·3,$$

or also

$$\sigma_0 = \frac{P}{a\sum_0^{24} f} = \frac{365.10^3}{50.2168} = 3·3;$$

$f$ represents any one of the forces $\overline{01}$, $\overline{12}$, etc. If we draw $\sigma_0$ corresponding to the centre of gravity of the section, and bearing in mind that the pressure is nil corresponding to the neutral axis, and that it increases lineally by moving away from it, we get the diagram of the pressure of the concrete (in the figure every cm. of ordinate corresponds with 1·2 kg. of pressure). Multiplying the ordinate of that diagram by 10, we get the pressure on the iron.

We have here:

Maximum unitary pressure on the concrete, 5·3 kgs. per sq. cm.

Maximum unitary pressure on the iron, about 53 kgs. per sq. cm.

This shows that the maximum stresses on the concrete and on the iron to which the house is subjected are by much inferior to the margin of safety, and thus there is a very large margin for other forces which have not been considered by us. By having in our calculation omitted the internal walls, we have possibly done something to the prejudice of the stability; but in view of that large margin there is no occasion to take any notice of that. A house planned like ours offers, therefore, favourable chances for resisting earthquakes.

In its general application, what we have said in Chapter X. holds also good for building structures in reinforced concrete, only, seeing that in the latter we have a much more suitable material than brick masonry, it admits of less compact and higher forms of houses.

We have assumed that the house was an elastic solid subjected to flexure, and we have neglected the shearing strain. That is permissible, and we refer in this respect to C. Bach, *Elastizität und Festigkeit*, 6th edition, p. 422 and following ones. There it is shown that the shearing strain may be neglected in view of the flexure whenever (as in our case) the point where $P$ is applied is distant from the base of the house not much less than one fourth part of the diameter of the section of the house, if that section is circular; or, when the section of the house is rectangular of height $h$, it is sufficient if the distance of $P$ from the base of the house be $h \leq .325\,l$. The house is then considered to be a solid fastened at the base.

By a correct mode of construction it will then be easy to unite rigidly the various parts of the house, *i.e.* the walls to the basement, the floors and the roof to the walls, and the latter between each other. It is advisable that every part of the house should be made of reinforced concrete, executed, of course, with all due care. Thus, care must be taken that expansion joints are left in the external walls, which, owing to their being exposed to sudden changes of temperature, might otherwise crack.

The mass of the reinforced concrete which forms the building ought to be reinforced, so that its various parts (*i.e.* the walls) will be able to resist stresses in their longitudinal direction also. The floors must unite the walls between each other rigidly, and therefore they ought to be specially reinforced with a view to the task imposed upon them. The same applies to the walls at their extremities.

Any possibility that, in the course of the operations, between the filling in of one layer and the next filling in, the various layers of concrete should not adhere perfectly together, must be absolutely avoided, because such an occurrence would jeopardise the principle of the monolith, which is the basis of this mode of construction. The old surface, after being well washed with cement-wash, must be sprinkled over with small bars of iron and then covered with a light layer of very rich cement mortar, after which the new concrete ought to be well rammed.

Generally speaking, care ought to be taken that the material used as well as the workmanship are of the very best. In the following chapter we reproduce the *Standard Rules for the Execution of Works in Reinforced Concrete* adopted in Italy. That, better than any other explanation, will serve to give an idea of the care and attention which have to be bestowed on the execution of constructions in reinforced concrete, and which must be even more rigorously observed in building houses destined to resist earthquakes.

It may be mentioned here that during the 1906 earthquake in San Francisco (California), and during the Calabro-Sicilian earthquake in 1908, it was clearly proved that reinforced concrete houses, *when well constructed*, are very well able to resist an earthquake.

## A MONOLITHIC HOUSE OF REINFORCED CONCRETE.

There does not seem to be any necessity for pointing out how the various parts of the house—roof, staircases, etc.—ought to be made. Building structures in reinforced concrete are, as such, already very well adapted to resist seismic shocks because they are elastic and monolithic.

It will suffice to counsel avoidance of all building constructions of a bold and jutting profile, as well as anything which might give extra weight to the upper part of the building. In the ordinary way of building, it is often preferred to erect, instead of continuous walls of reinforced concrete, a framework of that material, and to fill up the empty spaces with masonry. Such a system of mixed construction is far too liable to disintegration under the influence of the seismic forces to be worth recommending in the present case.

# CHAPTER XIV.

## STANDARD RULES FOR THE EXECUTION OF WORKS IN REINFORCED CONCRETE.

**Adopted by the Italian Association for the Investigation of Building Materials on [illegible]th May 1906, and by the Public Works Department by Decree dated 10th January 1907.**

Plans and Specifications—Quality of the Cement, and relative Tests—Quality of the Sand; of the Gravel—Aggregate—Mixing and Filling in of the Aggregate—Regarding Reinforcements—Tests of the Iron—Centerings—Dismantling—Load Tests—Rules for Static Calculations—Own Weight—Accidental Loads—External Stresses—Internal Strains—Calculation of the Pillars—Deformations—Safety Loads.

### GENERAL RULES.

1. EVERY work in reinforced concrete must be constructed upon the basis of complete specifications signed by an engineer.

The specifications must show the dimensions and the disposition of the concrete and of the iron, as well as the static calculations regarding them.

2. The execution of works in reinforced concrete shall only be entrusted to competent builders who can prove their competency by certificates granted to them under the regulations of article 2 of the General Conditions for Public Works.

3. The specifications must contain precise indications concerning the qualities and the properties of the materials to be used, the

ingredients composing the concrete, methods of construction, and of the dismantling and testing operations.

The quality of the materials must, if required, be proved by certificates granted by official testing laboratories.

## Quality of the Materials.

4. The cement must be exclusively Portland, slow setting, well seasoned, delivered in the original casks, and fulfilling the following requirements :—

(*a*) *Constancy of volume* to be proved by hot and cold tests, as a rule on specimens of the form of a cake or a ball.

(*b*) Minimum absolute density 3·05.

(*c*) Maximum residue on

a sieve of { 900 meshes. 2 per cent.
4900 ,, 20 ,,

(*d*) The setting of the standard paste of neat cement at a temperature of from 15° to 18° must not begin before one hour, nor terminate before five hours or after twelve hours.

(*e*) The tests of strength on machine-made specimens of standard mortar (1 part cement, 3 sand, by weight) must give at least the following results :—

| | | After 7 days of seasoning, the 6 last of which in fresh water. | After 28 days of seasoning, the 27 last of which in fresh water. |
|---|---|---|---|
| Tensile strength | kgs./cm. | 16 | 20 |
| Compressive strength | ,, | 180 | 220 |

All tests to be executed according to the rules of the Italian Association for the Investigation of Building Materials.

For works to be carried out in presence of salt water, the cement must, in addition, at the request of the manager of the works, be subjected to supplementary tests, as, for instance, to chemical analysis, to immersion tests and others.

5. The natural or artificial sand must consist of resisting and not excessively small grains; it must crackle in the hand and not leave traces of dirt. It must be free from any saline, earthy, vegetable, muddy, or powdery matter. In case it is not, it must be washed in fresh water until it fulfils these requirements.

6. The gravel must be quite clean and free from any extraneous substances, as also from saline, earthy, and friable matter. In case it is not, it must be washed in fresh water until it fulfils these requirements.

The gravel must be of such dimensions that it easily passes into the interstices between the casings and the iron reinforcements, and also between the latter. At all events, the maximum dimension must be considered to be 5 cm.

Whenever, instead of gravel, broken stones are used, they must be derived from compact rock, not from marls or any rock easily damaged by frost. They must be free from impurities or powdery, dusty matter. The size of the single stones must correspond with that prescribed for gravel.

7. The standard proportion of the different ingredients of the concrete shall be 300 kgs. cement per 0·400 cubic metre of dry and not compressed sand and 0·800 cubic metre gravel. In special circumstances a richer mixture may be demanded; in any case, however, the concrete must turn out full and compact.

The water for the mixture, like that for sand and gravel washing, must be clear, pure, and fresh.

The resistance to crushing by concrete of standard mixture after 28 days of maturation in a humid atmosphere, tested on cubes with sides of 10–15 cm. according to the size of its component parts, must not be less than 150 kgs. per sq. cm. For concrete of other proportions than the standard mixture the resistance to crushing, tested as above stated, must not be less than five times *the safety load* adopted in the calculation, with allowance of 10 per cent. for the mean breaking load.

8. For the reinforcement of the concrete preference shall be given to homogeneous iron obtained by the basic Siemens-Martin process. The iron must be smooth at the surface, free from swellings or blisters as well as from cracks and other interruptions of continuity.

The tensile strength, tested upon samples having an effective length of 20 diameters, prepared cold and in every respect conforming to the standard types adopted by the Italian Association for the Investigation of Building Materials, shall be comprised within from 36 to 45 kgs./mm.$^2$ The coefficient of quality, *i.e.* the product of the tensile strength per mm.$^2$ by the elongation (expressed in per cent of the length) of the sample, shall not be less than 900.

Whenever agglomerate or welded iron is used, it must be compact, malleable hot and cold, solderable, smooth on the external surface, and free from cracks; it must not show any burns, open soldering seams, or other interruptions of continuity.

The tensile strength, determined as above indicated, shall be at least 34 kg./mm.$^2$, with a minimum coefficient of quality of 480.

In addition to the tests regarding rupture by tension, the following bending tests may be demanded:—

### Bending Test for Homogeneous Iron.

A piece of iron heated to bright red heat and immersed in water at 28° centigrade must be able to be bent back upon itself, the diameter of the eye so made to be equal to the thickness of the iron, without any fault being produced.

### Bending Test for Agglomerate Iron.

The iron must be capable of being bent, cold, with the hammer round a cylinder the diameter of which is equal to six times the thickness of the iron, without any fault being produced.

All the above tests shall be carried out for every 100 pieces upon three samples taken, if possible, from the waste at the ends. If one of them does not stand the prescribed tests, two other samples shall be taken for every 100 pieces of the same material; if again one of them should not stand the required tests, the material shall be refused.

## RULES OF CONSTRUCTION.

9.—In preparing the mixture the various ingredients must be intimately mixed and uniformly distributed in the mass; the mixtures shall only be made in such quantities as are necessary for immediate use, *i.e.* before setting begins.

The materials composing the concrete may be mixed by hand or by machinery; whenever the importance of the work makes it possible, the latter process is to be preferred.

The preparation of the mixture shall be made on a paved ground, if possible near the place where it is to be used.

When no mixing apparatus is used, the cement and the sand must first be repeatedly mixed dry, and then this is to be further mixed with the gravel or the broken stones, and then the water added a little at a time; the mixing must be continued until the mixture assumes the appearance of scarcely humid earth.

10. When the casing for the filling-in of the concrete has been constructed, the iron reinforcements have to be placed in the positions prescribed for them by being tied with wire to the crossings and supported by provisional wooden struts.

Dirty, greasy, or considerably rusty irons must be thoroughly cleaned before being put into the work.

At the points of interruption the rods must be bound together for a length equal to 30 times their diameter, and their extremities bent over, or they can be united by formed hoops. Such interruptions must be placed each at a different section of the solid and occur in the regions of least stress. Weldings or solderings shall only be tolerated in places where the iron is not strained by more than 25 per cent. of the stress which the same can with all safety bear, provided that experimental tests carried out with three samples, chosen at discretion out of every hundred or part of hundred pieces, give good results.

11. Previous to the filling-in of the concrete the architect in charge of the works shall examine the position of the rods and see that it is strictly in conformity with the data of the specifications.

12. The concrete shall be filled in, immediately after the mixing process is finished, in thin layers well rammed with pestles of suitable form until the water comes out at the surface.

The concrete must completely surround the rods, and for that purpose the latter shall be, immediately before the filling-in, covered all round with a plastering of cement.

When a new layer of concrete is to be put on, and the preceding one is still quite fresh, its surface shall be bathed with water; but if the preceding layer has already commenced to set, its surface must be scraped off and moistened with a covering of cement, in order to secure the continuity of the structure. If the interval between filling in the two layers is of long standing, the washing process has also to be carried out.

13. In order to make sure that the concrete is always in conformity with the prescribed conditions, the architect in charge may, during the execution of the works, take some quantities out of it for the purpose of making test samples of them.

If the mean crushing load of such samples, after 28 days of maturation in a humid atmosphere, proves inferior by 10 per cent. to a stress five times stronger than the one which, according to the specifications, the concrete must be able to bear, the architect in charge of the works shall take such action as he may think fit.

14. It is absolutely prohibited to lay the concrete if the temperature is below the zero point, except in certain special cases, when special arrangements, to be approved by the architect in charge, will have to be made.

15. Proper arrangements shall be made to avoid the inconveniences caused by changes of temperature.

16. Until sufficient maturation, *i.e.* for a period of from 8 to 14 days, the works in reinforced concrete shall be periodically sprinkled over with water, covered with sand or linen, and kept

humid. In addition to that, they must be protected against the vicissitudes of the weather.

17. The centerings must be strong enough to withstand the weight of the structure itself as well as the vibrations produced by the ramming in of the concrete. It is also advisable to construct them in such a manner that at the moment of the first dismantling, while the necessary props remain standing, they can be removed without danger of damage to the works, the sides of the casings, and other less important parts.

In special cases it may be demanded that the wooden walls in contact with the concrete shall be perfectly planed and, if necessary, greased.

There must also be left in the wooden centerings some open joints of sufficient length to prevent the swelling of the wood produced by humidity disturbing the regular setting of the concrete.

18. During construction the works must not be subjected to the direct passage of the workmen and materials.

19. No attempt at dismantling shall be made before the concrete has reached a sufficient degree of maturation, and in any case 10 days must be considered to be the lowest limit for simple slabs up to about 1·[illegible]0 m. range. Works of greater range and of large dimensions shall remain longer in their centerings, the period to be stated in the specifications.

In seasons which are exceptionally unfavourable to the maturation of the concrete, the period prescribed for maturation shall be adequately prolonged. That holds good particularly for works which in the course of construction have been under the influence of frost. Before the dismantling of such works is started the whole period fixed for maturation has to elapse after it has been ascertained that thawing has been accomplished in the interio of the concrete.

During the removal of the centerings adequate arrangement have to be made to prevent the structure from receiving knocks shakings, or vibrations.

## LOAD TESTS.

20. In addition to controlling, in the course of the testing operations, the perfect execution of the work and conformity with the data of the specifications, the load tests may be proceeded with. For that purpose notice will have to be given in due time to the builder and to the contractor, advising them to be present.

The load test shall not take place in less than 60 days from the termination of the works. If the building structure, in the load test, can be loaded in the heaviest mode assumed in the static calculation, it will not be necessary to increase the intensity of the load. If, however, the tests are made with partial loads, the intensity of the test load must exceed that of the load in the calculation in such measure as will be decided in every individual case by the architect in charge of the works, who will make allowance for the benefit derived from the solidarity of the parts which have not been loaded. In any case that increase of the load shall not exceed 100 per cent.

Under the test load, permanent deformations showing themselves must not exceed 30 per cent of the total deformations. The elastic deformations shall be appraised according to the criteria indicated in No. 23, second paragraph. The total heights of incurvation for a structure with floors imbedded, even though it be imperfectly, at the extremities must never exceed one-thousandth of the span.

No building structure in reinforced concrete must be put in use, even temporarily, before the load test has been carried out; if the builder should make any use of it, it will be entirely at his own risk and responsibility.

## RULES FOR THE STATIC CALCULATIONS.

21. *Own Weight.*—As a rule, the own weight of the reinforced concrete, including the weight of the rods, shall be appraised at the rate of 2500 kgs./m.$^3$, except when, from special weighing operations, carried out for the building in question, other figures result.

22. *Accidental Loads.*—The accidental loads shall be determined by the same standards which are adopted for other kinds of constructions. Account shall be taken of possible dynamic actions by increasing the additional overburden by 25 per cent., or, in exceptional cases, even more.

23. *External Stresses.*—The external stresses shall be determined according to the ordinary theories of building science.

If statically undetermined constructions are in question, it is necessary, in order to compute the unknown forces, to assume, in appraising the geometrical entities of the transversal sections of the solids, that the superficial metallic elements are affected by coefficients ($m$) ten times greater than those of the elements of the aggregate $\left(m = \frac{E_f}{E_c} = 10\right)$, and that the latter are reacting even when they are stretched. If necessary, the normal modulus of elasticity of the *reinforced* concrete may be estimated to be

200 tons/cm.$^2$ If the percentage of the rods is less than 2 per cent., abstraction may also be made, in the above calculations, of its presence.

In case of inflected solids—such as are often met with in practice—it is often necessary, when calculating the sections in correspondence with the supports, to consider the perfect fixing down and the continuity of the beams; while for the central section of a beam, in the same cases, one may, when estimating the bending moment, start from the hypothesis, the moment at the supports to be only two-thirds of the above-calculated moment. In the absence of an exact computation of the condition of the fixing, *i.e.* imbedding, one may, for the central section, reduce by 20 per cent. the moment which would result from the hypothesis of simple supports at the extremities.

In the case of a slab reinforced by ribs, it shall be assumed that only a portion of the slab of which the size shall not exceed the smallest of the following dimensions, takes useful part with the ribs in the inflection, viz. distance between the ribs, centre to centre; twenty times the thickness of the slab; ten times the width of the ribs, one-third of the span of the ribs.

Slabs, reinforced in the two orthogonal directions and simply supported or fixed at all edges, may be computed like flags respectively supported or fixed all round.

24. *Internal Stresses.*—The ordinary methods of calculation shall be valid when the external force produces compressive stresses in all the elements of the transversal section of the solid (if in that section the superficial metallic elements are appraised according to the directions contained in No. 23).

If, however, (the superficial metallic elements being still valued is above), stresses of tension should be produced as well, abstraction shall be made from the tensile resistance of the concrete, and the axis which separates the reacting portion from the inert one, and the unitary stresses shall be determined by starting from the following principles :—

(*a*) Conservation of the plane sections.

(*b*) Proportionality of the stresses to the distances of the single superficial elements from the neutral axis.

25. *Calculation of the Pillars.*—The pillars shall, when the relation between the *free length of flexure* and the minimum transversal dimension exceeds 15, be calculated like solids loaded at the top, and allowance shall be made for any eventual eccentricity of the load.

The transversal bindings of the rods which reinforce the pillar must be carried out with the greatest care, and be *at least* so near to each other that they exclude the possibility of the *lateral deflection* of the rod considered as isolated.

26. *Deformations.*—As regards the calculation of the deformations, what has been said in No. 23, paragraph 2, concerning the valuation of the geometrical entities of the transversal sections o solids, and concerning the value of the modulus of elasticity $E$ ($E_f$ for the iron, $E_c$ for the concrete, $E_f = n \cdot E_c$), must be borne in mind.

27. *Safety Load.*—The safety load for concrete, at simple compression, shall not exceed one-fifth of the crushing load after 28 days of ageing, to be stated in the specifications, and, upon demand, to be proved by a certificate issued by an official testing laboratory.

No reliance shall be put upon the resistance of the concret to tension and shearing, considering that these stresses are exclusively borne by the iron reinforcement.

Homogeneous iron shall not be subjected to simple (*i.e.* without danger of lateral flexure) tensile or compressive stresses exceeding 1000 kgs./cm.², nor to shearing stress exceeding 800 kgs./cm.²

For agglomerate iron, the safety loads shall be four-fifths of those permitted for homogeneous iron.

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